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Dredging Work on the Panama Canal Slides

By W. G. Comber*

During the dry-excavation period of the Panama Canal work, progress was much delayed by the slides or landslips that occurred in different parts of Culebra Cut, principally between Empire and Paraiso from Mile 35 to Mile 39. These slides were of three general classes:

1. Cracks appear in the ground, running generally parallel to the axis of the canal and 100 to 1200 ft. back from the prism line. These cracks gradually widen until there is a rapid movement of the ground toward the prism.

2. Where the rock underlying the earth slopes steeply toward the canal, the slippery clays and broken detached rocks which overlie the rock flow down over this steep slope, which is lubricated by the heavy rain. The sliding material consists almost wholly of yellow or red clay and broken rock.

3. A combination of the above-cited classes.

When the water was let into the canal prism on Oct. 10, 1913, all the slides were under good control except that at Cucaracha. This defied the steam shovels and kept a constant stream of clay and rock pouring into the prism, almost like a glacier, turning over or covering up everything in its path. This slide is immediately south of Gold Hill, which is composed of hard trap rock. As shown by the accompanying map, the slide extended about 1600 ft. along the east bank of the canal and about the same distance back from the canal to the top of the ridge. The elevation above sea level of the limiting cracks was 475 to 600 ft. The material in the slide was soft rock and well-broken hard rock and a clay that has something of a floury consistency when dry and becomes a greasy mud when wet.

Fig. 1. View of the Culebra Slide, Looking South, Feb. 26, 1915
(Gold Hill on the left; Contractors Hill on the right)

By the utmost efforts of the steam shovels there was handled from this slide during July, 1913, 127,756 cu.yd.; August, 76,600 cu.yd., and September, 8100 cu.yd. No special cost sheets were kept of this work, but the division excavation cost for this period was 70.42c. per yd., and it is thought that the Cucaracha excavation cost was well above the average for the division.

When the water was let into the canal, the Cucaracha slide extended across the prism and filled it to a height of about 75 ft. on the west bank, and on the east prism line to 155 ft. above sea level, the water elevation at this time being 70 ft. above mean sea level.

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The plan for removing this slide involved the use of two pipe-line 20-in. suction dredges, three 5-yd. dipper dredges and the big ladder dredge "Corozal." It was first necessary to cut a channel through the slide sufficient to float the dredges into their proper positions. The north part of the slide consisted largely of hard trap-rock masses broken off the south side of Gold Hill. It was planned to use the dipper dredges at this end. The suction dredges were to pump the softer material off the south face of the slide through a 20-in. electrically driven relay into the Rio Grande Valley over an elevation of about 140 ft. mean sea level, or 70 ft. above the water level in the canal.

Work was begun by excavating with two dipper dredges on the north face of the slide and with one suction dredge, No. 85, on the south side. The latter was withdrawn in a few days, as its pump was double suction, and choked very badly. The dredge "Corozal" also worked on the south side, but was withdrawn in a short time, as this dredge, being of the deep-draft type, was unable to cut its own flotation in this class of material. In this dilemma the old French ladder dredge "Marmot" was brought up, and, being ably handled by the old French foreman, Laurent Roquebert, is entitled to the credit of first opening a channel through Cucaracha, the dredge working 24 hr. per day, seven days in the week. A channel about 25 ft. deep and 140 ft. wide was opened by Dec. 8, 1913, and all dredges were passed to their proper stations. The two open pump suction dredges No. 86 and No. 4, which were on the north side of the slide, were passed to their station on the south side, and the five-yard dipper dredge "Cardenas" was passed to the north side. After the initial channel was cut the "Marmot" was also worked whenever possible on the north side.

It was expected that the new 15-yd. dipper dredges "Gamboa" and "Paraiso" would reach us early in 1914, but various troubles delayed their completion, and the first of these dredges, the "Gamboa," was put at work Apr. 1 and the "Paraiso" June 7. At this time there was in motion from the east bank a hard trap dike, extending from an elevation of +100 ft. mean sea level to the bottom of the canal. The three-drill barge "Teredo" was brought up to undermine and blast this obstruction, which was successfully done; but unfortunately, through an accident, the "Teredo" was blown up and sank in the middle of the channel 25 days before Aug. 15, 1914, the date announced for the passage of commercial steamers through the canal. After an examination by divers, as the "Teredo" was nearly cut in two amidship, it was determined to cut the wreck entirely in two, which was done by passing a line of 2½-in. hose filled with 60% dynamite and exploded by a battery.

No crane then on the Isthmus was of sufficient capacity to lift the halves of this wreck, but they were hauled a distance of 2000 ft. by lifting one end of each piece with the main hoist of the 15-yd. dipper dredges. Three tugs were hitched to the dredge and the wreck hauled away, one piece on the north and one on the south side of the slide, and the channel was opened for shipping on the date set. These halves of the wrecked "Teredo" were afterward lifted by the 250-ton crane "Ajax."

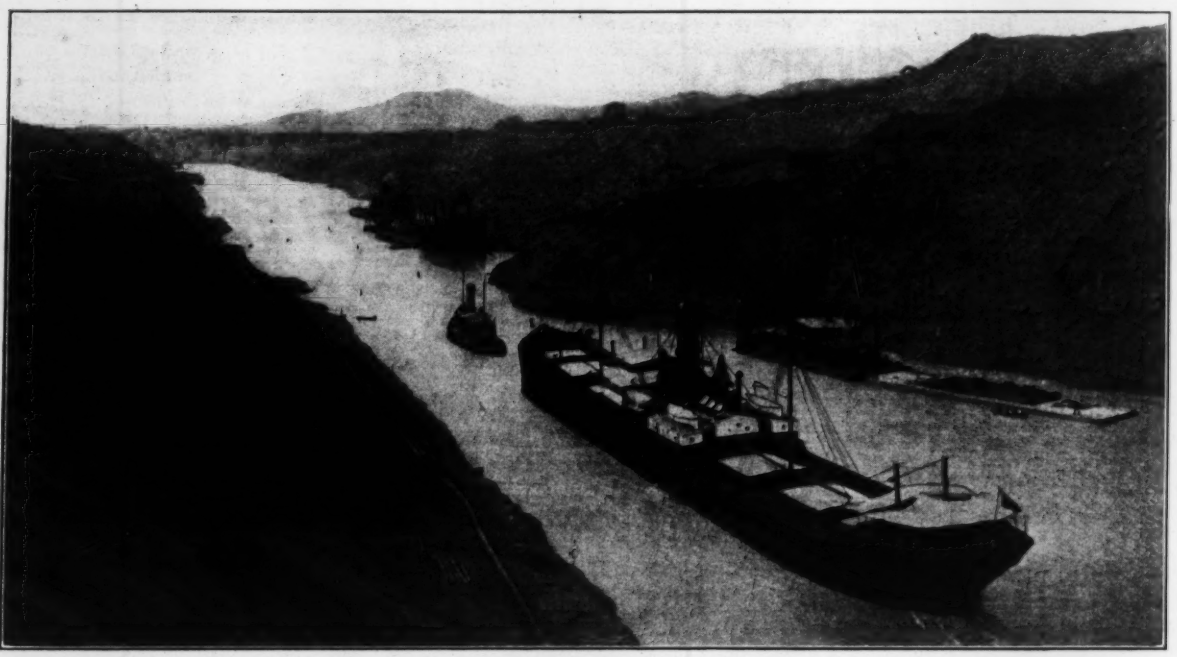


FIG. 2. VIEW LOOKING NORTH AT CULEBRA SLIDE, FEB. 26, 1915
(Note the buoys guiding commercial vessels past the slides)

Wet excavation and cost per month in Cucaracha slide are appended up to and including Oct. 13, 1914, as follows:

Month and Year	Output, Cu. Yd.	Cost	Unit Cost
October, 1913	3,805	$18,332	$4.8180
November, 1913	68,979	30,586	0.4434
December, 1913	321,193	123,465	0.3844
January, 1914	291,574	151,205	0.5186
February, 1914	287,223	164,394	0.5724
March, 1914	327,796	187,588	0.5722
April, 1914	387,708	180,478	0.4655
May, 1914	352,659	171,194	0.4854
June, 1914	381,438	192,589	0.5049
July, 1914	308,804	160,771	0.5206
August, 1914	356,273	163,980	0.4603
September, 1914	306,978	151,211	0.4926
October, 1914	157,829	66,288	0.4200
Total	3,552,259	$1,762,089	$0.4960

At this date, Oct. 14, Cucaracha was cleaned up to a full width of channel with a least depth of 32 ft., it being necessary to break up the hard rock in the bottom before it could be dredged to the full depth.

mean sea level, gave way. It is roughly estimated that there is in the neighborhood of 5,000,000 cu.yd. in this

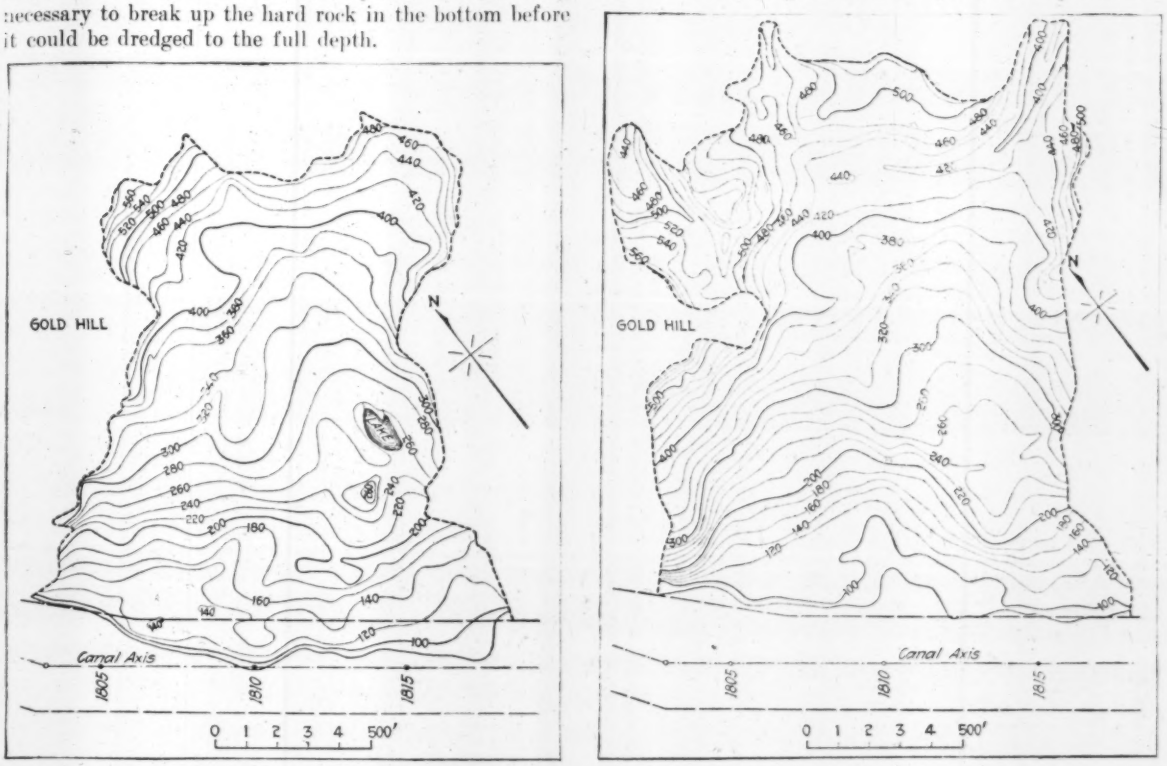


FIG. 3. MAPS OF CUCARACHA SLIDE (HEAVY DASH LINES DEFINE LIMITS OF SLIDE)

(Feb. 14, 1914. Material removed by dredges, 844,111 cu.yd.) (Oct. 1, 1914. Material removed by dredges, 3,552,259 cu.yd.)

No further movement of Cucaracha was noted until Jan. 14, 1915, when the loose superficial rock and clay over the underlying rock slopes gathered enough force under the changed conditions of the dry season to put about 100,000 yd. into the canal, with a shoaling up of the east side of the channel from about elevation +95 ft. on the east prism line to elevation +73 ft. on the center line, or only 14 ft. of water. The ladder dredge "Corozal" was brought up from the Pacific side, and the west side of the channel was again cleaned up. This work was done without stopping the passage of commercial shipping; and this part of the channel is in this condition today except that the hard rock has been broken to grade +40 over nearly the whole of the prism at the slide, while the dredges have been engaged on more urgent work at Culebra slide.

NEW CULEBRA SLIDE

On the afternoon of Oct. 14, 1914, an area of the east bank immediately north of the hard trap-rock face of Gold Hill, 1600 ft. long and about 1200 ft. wide, with an elevation at the back of the slide of from 350 to 450 ft. above

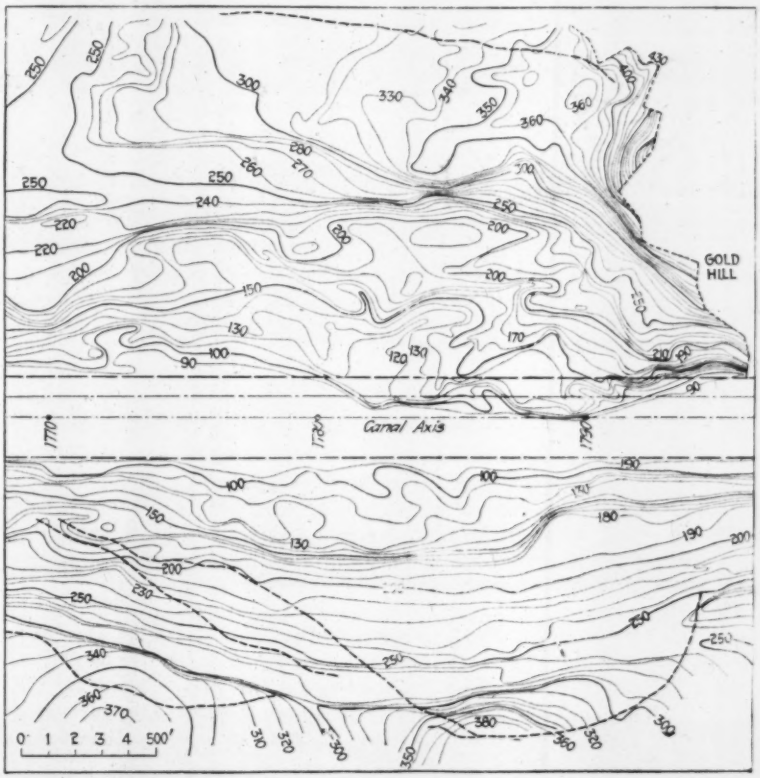


FIG. 4. MAP OF EAST AND WEST CULEBRA SLIDES, JAN. 1, 1915

slide. The initial movement of this land slip entirely filled the canal prism, except for a winding, tortuous passage along the west bank, with about 16 ft. of water. To avoid being bottled up, the dipper dredges, tugs and barges which were south of the slide were passed to the north side while the movement was going on. All material excavated by the dipper dredges has to be carried north into Gatun Lake and dumped into deep water, away from the channel. The 15-yd. dipper dredges were at work on the north side the same evening, and by 6 a.m. of Oct. 15, 9510 yd. of material had been taken out. In the following six days 130,082 yd. were excavated and commercial navigation was resumed. However, on Oct. 31 passage of ships was stopped for two days on account of a new movement. Since that time navigation has been uninterrupted, except that very heavy draft ships have been held up from 1 to 6 days to get sufficient depth of water.

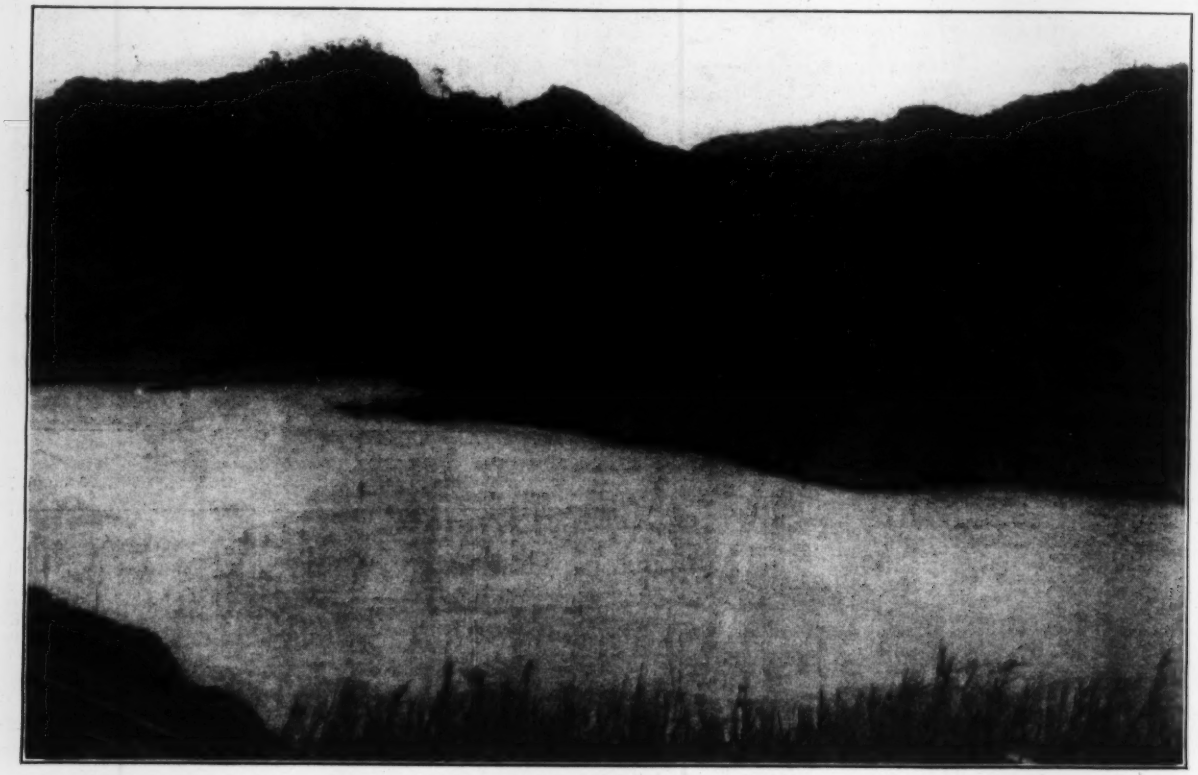


FIG. 5. VIEW OF CUCARACHA SLIDE, FEB. 26, 1915

The material in this slide is chiefly soft slate-colored rock, well broken in its passage to the canal, and in its removal presents no particular difficulty to the 15-yd. dredges, except the unavoidable delays occasioned by the cramped quarters in which all the dredges are working and the necessity for passing the ships.

There have been taken out of this slide the following yardages, by months: October, 346,368; November, 484,697; December, 500,160; January, 572,608; February, 573,846. The unit cost was $0.345 per cu.yd.

As there is a pronounced crack extending longitudinally along the canal well back of the present break at this slide, it is thought that some time in the future more material than now shows will come down; but owing to its distance from the canal, it is thought that it can be handled without hindrance to navigation.

As the depth in the channel varies from day to day, and the commercial shipping must be passed without undue delay, the following plan is being followed: The depth required by the deepest ship is determined at the earliest date possible, and as soon as this draft is known the available channel is dragged by suspending from a skiff by wires a 15-ft. long 2-in. iron pipe 2 ft. below the required depth. Four of these drags are in use, and every inch of the slide channel is covered. The obstructions found are marked with a small buoy, one or more dipper dredges being detailed to remove these obstructions to below the grade. When this is completed the dredges are moved away and the right-hand side of the slide channel looking north is marked by small black buoys and the left-hand side by small red buoys. The slide channel is opened for traffic at 2 p.m. each day as nearly as possible, the ships being in waiting as close to the slide as may be convenient. After the passage of the shipping the buoys are removed and the dredges resume their work until the clearing of the channel the following day.

A survey of the slide channel is made every morning and plotted, the map being in the hands of the dredge foreman and superintendent by 11 a.m. each day. A sample of the daily map is appended.

THREATENED SLIDE AT CULEBRA WEST BANK

The west bank at Culebra was very troublesome during the dry-excavation period and for a long time compelled the operation of steam shovels on various levels outside the prism for the purpose of reducing the slope and relieving the pressure. This work was discontinued June 7, 1914.

Before the above described work was stopped, and since that time, longitudinal cracks have been opening up at various elevations, and on Dec. 9, 1914, a survey was made

of these cracks and various stations were established, marked by level stakes on each side of the cracks, and observations made each day with a view of determining the rate of movement. This work was extended to the east side also, to the cracks mentioned back of the new Culebra slide on the east bank. The accompanying map indicates the location and elevation of these cracks.

The output shown by the dredges at this work is not phenomenal, except that the material handled is almost entirely broken and ground-up soft rock, and the work is carried on 24 hr. in a day and seven days in a week, with only the necessary stoppages for repairs.

Manganese-steel rock buckets of 10-yd. capacity are used on the 15-yd. dredges, and all material except the soft earth in the south part of the slide is loaded into barges and dumped in deep water in Gatun Lake.

The Cucaracha slide was at least temporarily beaten at the time the new Culebra slide gave way, and it is hoped that the latter slide will be removed before the threatened break on the west bank at Culebra occurs.

Street-Name Signs

The marking of streets and the numbering of houses are very proper functions of a city engineering department, and some cities have progressed so far as to place these under the jurisdiction of the city engineer. In other cities the work is under a bureau of highways, a street commissioner, or a special bureau; on new streets it is commonly done by real-estate companies.

Information received from eight cities—Chicago, San Francisco, Boston, Albany, Cleveland, Kansas City, St. Paul and New York—shows that only four have adopted a uniform standard practice of establishing street-name signs. The common method seems to be to attach signs to any post (telephone, lamp or trolley) at or near the street intersection. In parts of Kansas City, St. Paul and New York it is common practice to place the glass signs on street lamps. This is generally done in residence streets, which, being darker at night than the business streets, make an illuminated sign desirable. Such signs are usually four-way signs, the street names being parallel to the street they refer to on both sides of the lamp. In St. Paul there is a uniform practice of placing such signs, in residence districts and in the downtown streets, on lamp-posts, independent of the lamps. Two blue enamel signs lettered in white on both sides are the commonest practice, although in St. Paul there are four signs to a pole.

Few cities erect special posts for signs, except in the absence of any other pole. Cleveland has a notable practice in this respect, the signs upon some residence streets being placed on ornamental cast-iron posts (Fig. 1). The post is a casting 2 or 2½ in. in diameter, and is erected in the space between the curb and the sidewalk and carries a four-way sign. At street intersections of other than 90° a swivel bracket is used which permits the signs to be set at any angle to each other. The posts are about 8½ ft. from ground to cap, and the cap is 18 in. from base to tip of acorn. These posts and signs are not patented.

Only two cities—Boston and Kansas City—have street names in the sidewalk, but in neither city is such a method general. Fastening the signs to corners of buildings is common practice in Boston, Albany, Kansas City and San Francisco; but this is probably the least satisfactory method. As is the case in Boston, these signs are frequently hidden by advertising signs, and they are difficult for the eye to find under any circumstances. In the Chicago business district the street names in some instances are painted vertically in white on trolley and electric-light poles.

Only in exceptional instances, as in the case of the intersection of very wide streets or boulevards, are street-name signs erected on all four corners. In Chicago the signs are placed at one corner only, except on what are known as base-line streets, from which intersecting streets are to be known as N. or S., E. or W., where there are two signs on diagonally opposite corners. In Chicago the N. W. and S. E. corners are preferred; in New York, the S. W. and N. E. corners. New York is the only city to try a sign which gives both street names on each board (Fig. 2), the parallel street being shown below in large letters and the crossing street in smaller letters above. This sign offers equal convenience to vehicular passengers and pedestrians.

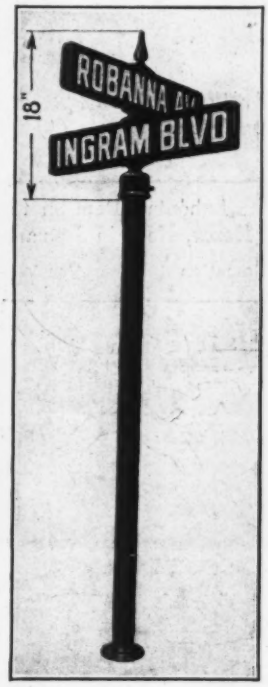


Fig. 1. Cleveland, Ohio

Fig. 2. New York City

Street-Name Signs

Conclusions—The preceding summary of the street-marking practice of eight cities shows that progress is being made to improve this feature of municipal orderliness. It shows that the problems of marking the business and residence streets are essentially different. Only Boston exhibits an apparent disregard for the need of complete and respectable street-marking. Here the district foremen of the street department put up signs wherever they can find a place, frequently neglecting to put up or maintain any sign whatsoever. Signs on the corners of buildings are undoubtedly the poorest practice, and signs on special posts or on corner lamp-posts like those shown in the accompanying illustrations are probably the most satisfactory. Two posts to a street intersection are universally deemed sufficient.

Lahontan Dam, Truckee-Carson Irrigation Project, Nevada

By D. W. Cole*

SYNOPSIS—An earth embankment 124 ft. high stores 290,000 acre-feet of water for irrigation at a cost of about $1,500,000. Among the unique features are (1) twin spillways 250 ft. long at either end of the dam with a combined normal discharge capacity of 30,000 sec.-ft., converging on a circular pool 230 ft. in diameter; (2) a reinforced-concrete outlet tower, with two cylindrical hydraulically balanced valves 8 ft. in diameter having a combined discharge capacity of 2000 sec.-ft. at the lowest stage of storage, with air ducts to relieve the tendency to vacuum and vibrations; (3) an up-to-date electrically operated construction plant; (4) a sand-cement plant.

The virtual completion of the Lahontan Dam on the Carson River, near Fallon and Hazen, Nev., in January, 1915, makes timely a general description of the design and construction of this unique dam, particularly as many engineers from the East may wish to visit it going to or coming from the Panama-Pacific Exposition. Those who wish to improve this opportunity will stop off at Hazen, Nev. on the Union-Southern Pacific or Overland Route and take automobile or team to Lahontan, seven miles to the south.

*Project Manager, U. S. Reclamation Service, Truckee-Carson Project, Fallon, Nev.

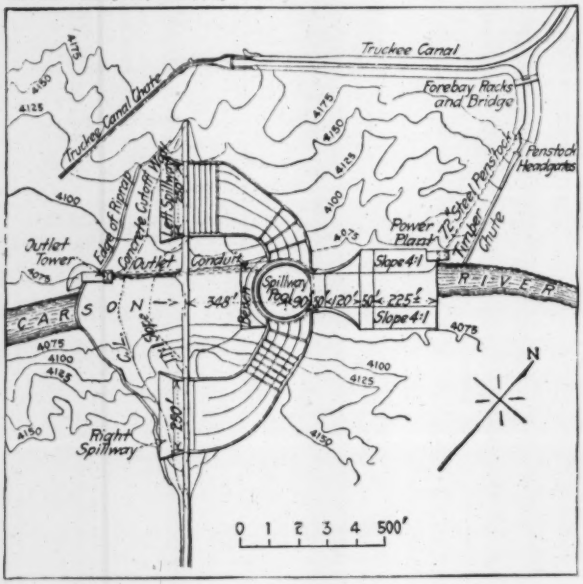


Fig. 1. Map of Lahontan Dam Site

Purpose and Location of the Lahontan Dam

The object of the Truckee-Carson Project of the United States Reclamation Service is the reclamation for agriculture by irrigation of about 200,000 acres of desert land. This involves the storage of flood waters from the melting snows of the Sierras. The dam forming the main storage reservoir in the Carson River branch of the drainage system was named after the ancient Lake Lahontan.*

The location of the dam was fixed at practically the last available site where the river emerges from the foothills, and it was at this point also where it was possible to place the discharge of the Truckee Main Canal so that waters from this source could be discharged into the reservoir

*A companion to Lake Bonneville, of which the Great Salt Lake is a remnant. The ancient Lake Lahontan was named for Baron La Hontan, an early explorer of the Mississippi Valley. The Lahontan Reservoir constitutes the largest storage unit in the Carson River branch of the drainage system.

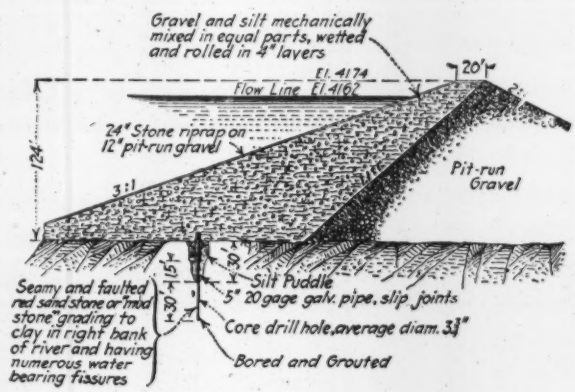

Gravel and silt mechanically mixed in equal parts, wetted and rolled in 4" layers
El. 4174
20'
Flow Line El. 4162
24" Stone riprap on 12" pit-run gravel
124'
3:1
Pit-run Gravel
Silt Puddle
5" 20 gage galv. pipe, slip joints
Core drill hole, average diam. 3½"
Bored and Grouted
Seamy and faulted red sandstone or mud stone grading to clay in right bank of river and having numerous water bearing fissures

Fig. 2. Section through Lahontan Dam

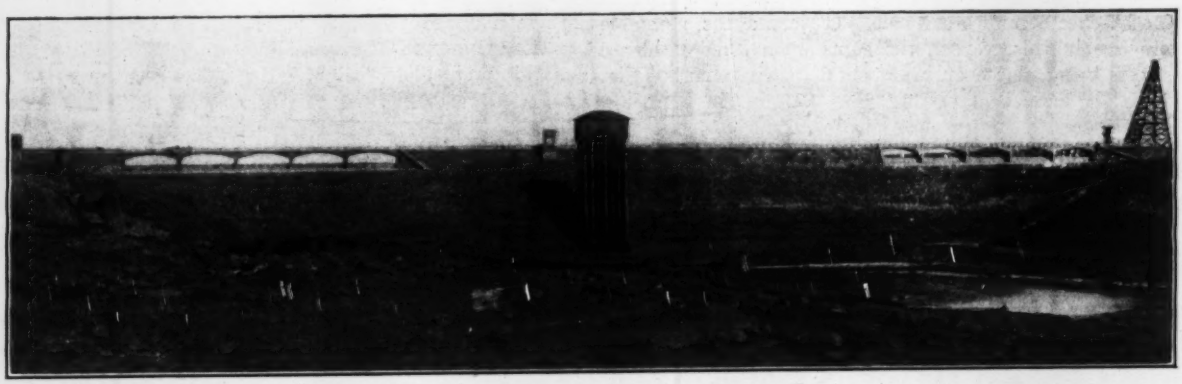


Fig. 3. Upstream View of Lahontan Dam, Showing Twin Spillways and Gate Tower

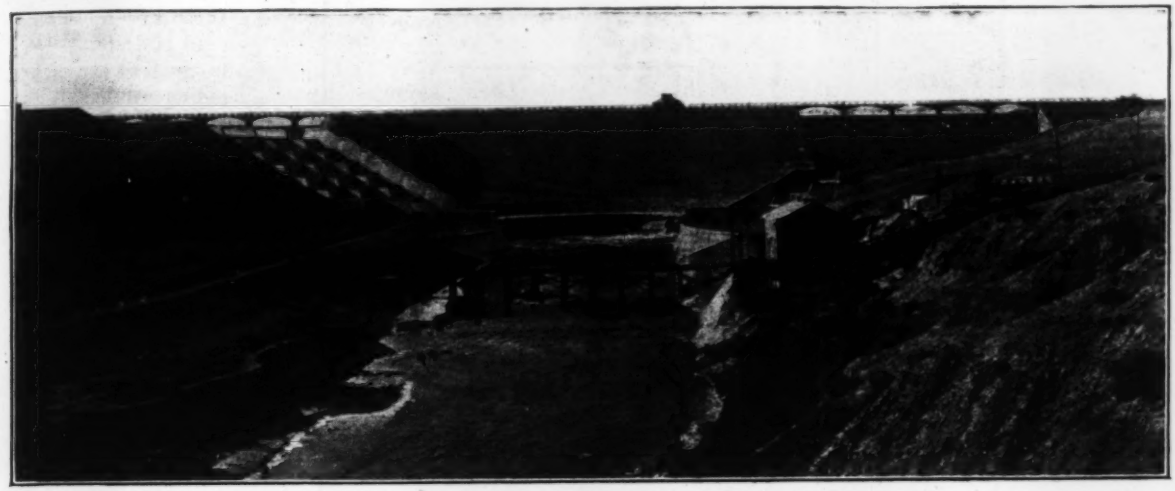


FIG. 4. DOWNSTREAM VIEW OF LAHONTAN DAM

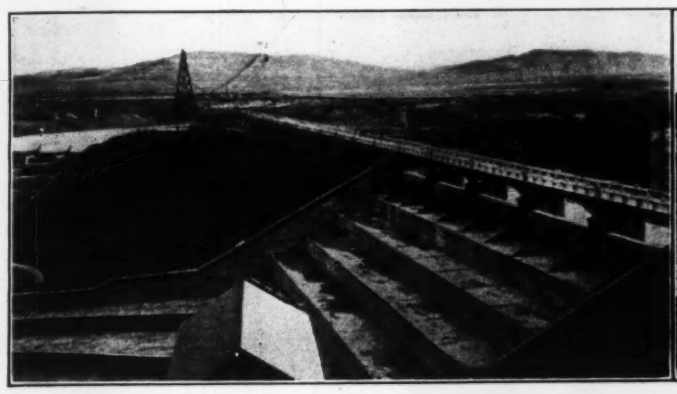


FIG. 5. LOOKING ACROSS THE DAM

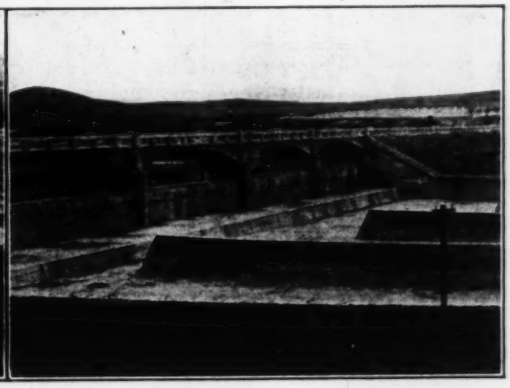


FIG. 6. UPPER PART OF RIGHT SPILLWAY

FIG. 7. LEFT SPILLWAY AND SPILLWAY POOL

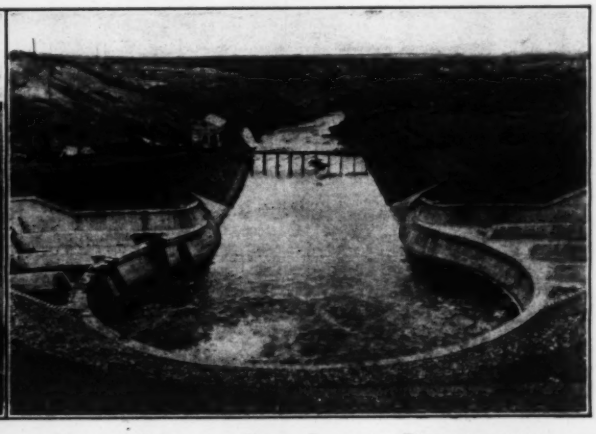


FIG. 8. SPILLWAY POOL, LOOKING DOWNSTREAM

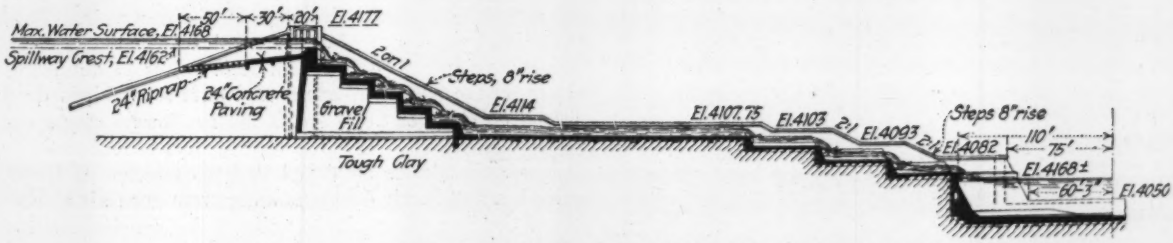


FIG. 9. LONGITUDINAL SECTION THROUGH LEFT SPILLWAY OF LAHONTAN DAM

formed by the dam. The Truckee River is a larger stream than the Carson, with a more prolific drainage area, and effective storage for it is found in Lake Tahoe near the summit of the Sierras. Waters of the Truckee River are diverted into the main canal of 1200 sec.-ft. capacity and 31 mi. in length, terminated by a concrete chute in the reservoir immediately above the Lahontan Dam. The storage capacity of the reservoir is 290,000 acre-ft. The total cost of the reservoir was approximately $1,500,000.

Type and Dimensions of Dam

The type of structure was the subject of long and studious consideration. At the inception of the project a masonry dam of gravity section was planned, but subsequent borings and examinations of the site led to the adoption of the earth-embankment type.

The choice was determined by the relative instability of footing for masonry construction and by the necessity for ample spillway capacity to take care of floods up to 30,000 sec.-ft. The character of the foundation was such that it seemed inexpedient either to build or to spill the flood waters over a standard masonry section, and there was no sound ledge rock which might be availed of for a spillway site. The formation at this dam site was of a soft and friable material called "mud stone" by the geologists and resembling red sandstone in texture. The rock was not continuous across the gorge and was intermingled with clay formations in a very irregular manner due to faulting and folding of the strata.

Twin Spillways—The adoption of the embankment type involved the provision of a spillway for carrying large floods over an earth structure exceeding 100 ft. in height—a quite unusual requirement. The design of this spillway was carefully studied and experimented with by the consulting engineers, and many preliminary drafts were abandoned when tested by the local conditions.

Finally it was determined to build the main dam of standard embankment cross-section, flanked at either end by spillways each 250 ft. long, giving, with 6-ft. depth on the crest, a discharge capacity of approximately 30,000 sec.-ft., which is 50% above the greatest known flood. Further factors of safety are found in the short duration of floods in this stream, which will spread out over the 12,000 acres of reservoir before producing an appreciable rise. The water on the spillways, moreover, can rise to 12 ft. before overtopping the embankment section. Therefore, it is estimated that floods of even 50,000 sec.-ft. would be spilled without damage to the structure. Fig. 1 is a plan and Fig. 2 a section of the dam. Figs. 3 to 8 are views of the structure, completed with the exception of the lamp-posts and the footbridge to the gate tower. All of these views were taken on Jan. 20, 1915.

The duplicate spillways (see Fig. 9 for longitudinal section of left spillway) converge below the dam into a central pool having a diameter of 230 ft. and an area of nearly an acre. The energy of the two waterfalls will thus be dissipated and absorbed in the pool before final discharge with moderate velocity into the river bed of clay, which, for some distance downstream, is amply protected by paving. The bottom of the pool is paved with concrete slabs 4 ft. in thickness and underdrained into the inspection gallery in the downstream pool wall. In fact, the entire spillway channels are underdrained with open-jointed vitrified pipe laid in gravel-filled trenches. Similar drains extend under the base of the embankment in the river bed nearly to the center line so that the waters of percolation will find an outlet to the inspection gallery built into the encircling wall of the pool. The entire spillway system is built of heavy concrete floor slabs and walls partly reinforced and founded upon the broken formation of mingled rock and clay with a view to taking up the shocks of the waterfalls and preventing all erosion of the naturally unstable material of the side. The left spillway is perhaps unique in being built in cyclopean steps upon buttress walls placed within the embankment, thus constituting virtually a concrete-faced-embankment spillway.

Outlet Tower and Gates—Another interesting feature is the outlet tower and the gates for controlling the flow through the outlet conduit under the base of the dam and discharging it into the spillway pool. The tower (Fig. 10) is of massive reinforced concrete in which are set 12 gates, at two different elevations, through which water from the reservoir is taken into the central chambers and discharged into the conduit by means of two cylindrical hydraulically balanced valves of novel design at the bottom of the chambers. Two smaller sluice gates in

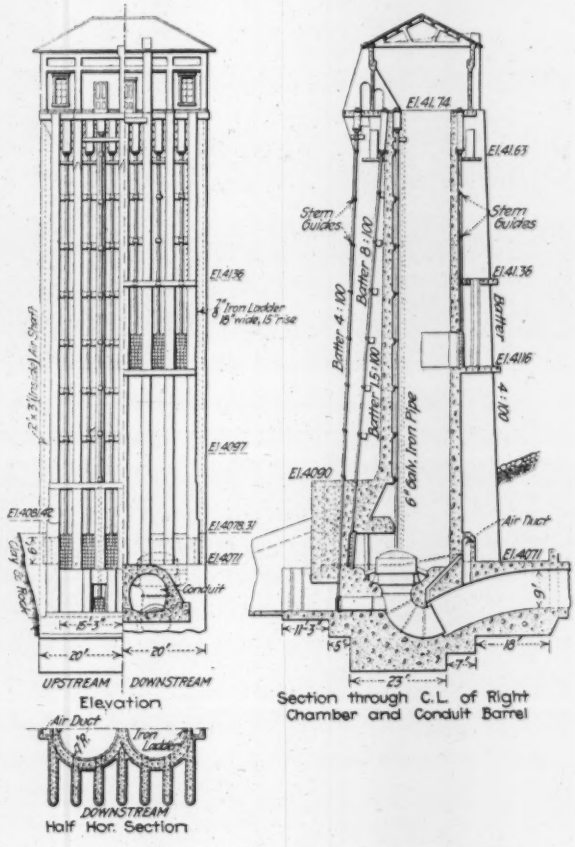


Fig. 10. Outlet Tower of Lahontan Dam

the side of the tower under the cylindrical valves also serve for the ultimate drainage of the reservoir. The main inlet gates are of standard sliding pattern, each 3x8 ft. in area. The two cylindrical valves in duplicate chambers are 8 ft. in diameter and have a combined discharge capacity of 2000 sec.-ft. at the lowest stages of storage.

A peculiar device employed in the operation of these gates is a 2x3-ft. air duct extending from open air at the top of the tower through the walls of the structure to the top of the outlet conduit at the entrance of the 7-ft. diam-

eter jet of water below each gate. These ducts relieve the tendency to vacuum and obviate the consequent vibrations by a liberal supply of air to the rapid and free discharge of water through the conduits.

All of the gates are operated by hydraulic oil pressure in cylinders set in the gatehouse floor under a pumping system electrically operated.

Access to the gatehouse and controlling mechanism is secured by a suspension footbridge extending from the top of the dam to the floor of the house at the top of the tower.

CUTOFF WALL—An important feature of construction was the concrete cutoff wall (Fig. 2) in the bed of the river beneath the embankment. The bottom of this wall extends to a depth of from 30 to 60 ft. below the original surface, with additional penetration of the foundation strata by means of grouting the fissures in the rock and clay. This grouting process and the results obtained as determined by test were quite fully described in ENGINEERING NEWS of Apr. 3, 1913, and subsequently in a paper by H. S. Rands before the American Society of Civil Engineers.

THE DAM—The embankment itself was composed of two portions. The upstream portion was of a prepared mixture of gravel and soil or silt in nearly equal parts, which was found by careful and repeated experiments to approach the greatest density and water-tightness. The downstream portion was built of pit-run gravel to afford a ready and harmless escape for water percolating through the upstream water-tight section.

The reservoir slope is protected by 12 in. of coarse gravel next to the embankment, with an outside coat of 24 in. of roughly hand-placed stone riprapping.

The cross-section (Fig. 2) of the dam has a top width of 20 ft., an upstream slope of 3 to 1 and a downstream slope of 2 to 1, the latter being broken 12 ft. above the pool wall by a circular berm 10 ft. in width, concentric with the pool.

The dam is about 1300 ft. in length over all. The roadway at the top is 12 ft. above the flow-line, or crest of weir, and 124 ft. above the original river bed. The roadway over the top of the dam is carried across the two spillways by means of reinforced-concrete arches in 50-ft. spans and 5-ft. rise. A concrete railing guards the roadway and carries electric-wire conduits for lighting the structure, the gatehouse and the roadway at night.

An earth dike three-fourths of a mile long and about 4 ft. high, level with the top of the dam, extends from the dam southward over the mesa to the foothill.

CONSTRUCTION PLANT

The structure was built under thoroughly modern conditions. The fall of the Truckee canal into the Carson River afforded excellent opportunity for the installation of a hydro-electric plant of 1500 kw. capacity, which subsequent to the construction of the dam has been leased to an electric-power operating company. With this electric-power plant as a basis the construction plant for the dam was designed to make the fullest use of the cheap power. The only steam power employed was for traction engines for rolling the embankment, small locomotives for borrow-pit hauling and for the operation of two small steam-shovels which were in use before completion of the main power plant. All other portions of the plant, including the main borrow-pit shovel, the dragline excavator, a number of belt conveyors, a sand-cement plant, the cableway, pumps, derricks, blowers and drills, were operated by electric motors. Probably the first fully successful electric shovel was employed on this work and handled the 500,000 cu.yd. of gravel at a cost very much below what a steam

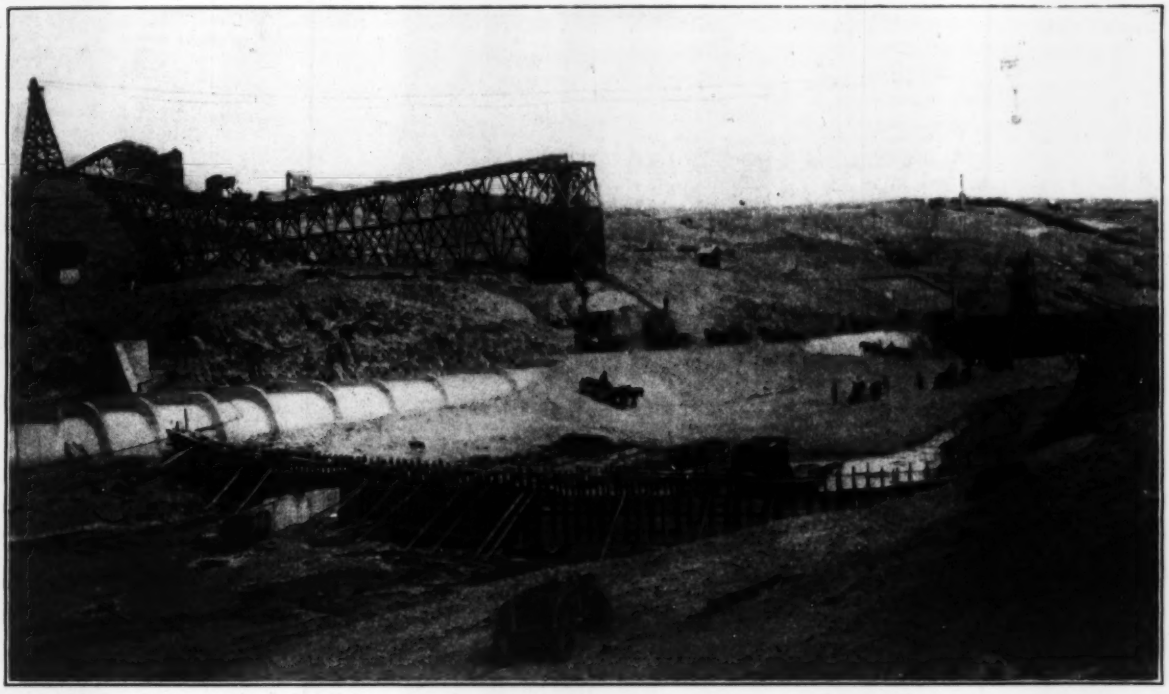


FIG. 11. PLACING EMBANKMENT MATERIAL IN RIVER BED, LAHONTAN DAM

(The high trestle supported the belt conveyor which brought embankment material for distribution by wagons. The outlet conduit is in the middle of the view, and at right angles to it is the cutoff wall)

shovel would have shown at the local prices for coal. All of the electric equipment was highly effective, convenient and satisfactory throughout.

A novel application of the belt conveyor was employed for delivering the entire quantity of embankment material, about 700,000 cu.yd. This conveyor of 925 ft. in length by 30 in. in width extended overhead and to the middle of the main embankment (Fig. 11) from the storage bins for gravel and soil, which were filled by means of dump-car trains from the borrow-pit.

The peculiar advantages of this conveying system were found not only in reduced cost of delivery but in the facilities afforded for securing a perfect blending of the embankment materials to form the water-tight mixture. This mixture as well as the gravel portion of the dam was delivered by dump wagons, loaded through chutes from the conveyor, and thoroughly rolled in 3-in. layers by 10-ton traction engines.

Other belt conveyors were used for delivering concrete gravel to the screening plant and to the mixing plant. The concrete-mixing plant was also of modern design, employing mechanical devices for proportioning, assembling and mixing the ingredients with a minimum of manual labor.*

A sand-cement plant for mixing and regrinding portland cement with local sands was also electrically driven and equipped with a conveyor system. A large portion of the concrete in the dam was built of this form of cement.

The 70,000 cu.yd. of concrete in the spillways and other structures was delivered by the main cableway of 1600 ft. span with a transverse movement of 750 ft. and 15 tons capacity, operated by a 300-hp. electric motor. In detail, the concrete was placed by a local adaptation of the spouting method. The cableway also delivered all of the slope paving and much of the screened gravel and other materials used in drains and elsewhere. The cableway, furthermore, was almost indispensable for moving heavy equipment and doing other kinds of work in connection with the construction.

The construction program extended over a period of about four years, ending in January, 1915, and was carried out in accordance with a predetermined schedule and at a cost of about $50,000 within the preliminary estimate. The work was done by the administration forces of the United States Reclamation Service directly in charge of the Truckee-Carson Project Manager, acting through a resident engineer and a superintendent of construction located at the model camp near the site of the work.

*The use of sand-cement on Reclamation Service projects was described in "Engineering News," Feb. 15, 1912, p. 305; Mar. 20, 1913, p. 562; June 19, 1913, p. 1271.

Switching Locomotives Driven by Gasoline-Engine Power

There has been a steady and rapid development, during the past three or four years, in the application of internal-combustion engines using gasoline or kerosene as fuel to the driving of light locomotives. The early development was in the field of locomotives for use for mining purposes and around industrial plants. These machines were of comparatively small size and power. Recently larger machines have been built, especially designed for switching service.

From present appearances the time is near at hand, if indeed it is not already here, when the gasoline locomotive will displace the steam locomotive for switching purposes in very many places. True, the gasoline locomotive has not as yet attained such size and capacity that it can replace steam locomotives in the operation of a large city terminal yard. There is, however, an enormous amount of work done by switching locomotives around country stations and in the yards and shops of manufacturing plants and other industries. If this work is done by steam locomotives, either a switching locomotive must be kept fired up all day to do perhaps not more than one or two hours' work, or else the local freight train must be delayed so that its locomotive can do the work. Either of these plans is extremely costly for the amount of work done.

Gasoline locomotives are now available which can do three-fourths or possibly nine-tenths of this work as well as a steam locomotive. The gasoline locomotive is always ready for use when it is needed, without the delay of getting up steam, and uses fuel only while it is actually at work. The fuel it uses is, of course, far more costly

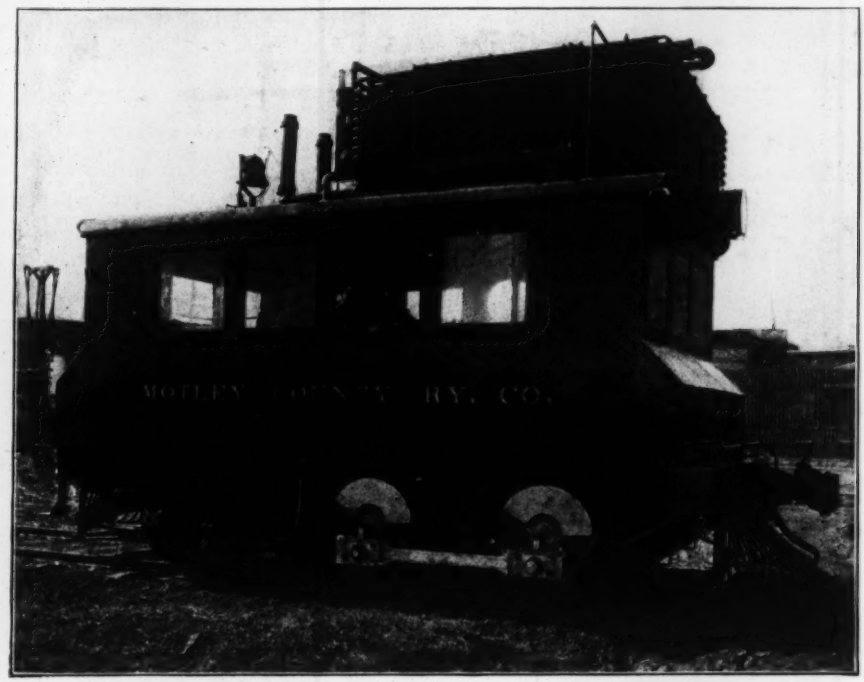


FIG. 1. GASOLINE-ENGINE-DRIVEN SWITCHING LOCOMOTIVE OF 300 HP. FOR A TEXAS RAILWAY
(Built by the McKeen Motor Car Co., Omaha, Neb.)

than the coal burned by the steam locomotives, but the quantity it uses is so small that the actual fuel expense may be less than that of the steam locomotive.

Of course, one great advantage of the gasoline locomotive over the steam locomotive for switching service is that it is a geared machine. Nearly all switching service is done at low speed, and ought to be so done to reduce damage to rolling stock. The wonder is that none of the steam-locomotive builders has ever put on the market a geared switching locomotive for light work, built along the lines of the present gasoline locomotive. A very large market might have been found for such a machine had it ever been developed.

Certain features of advantage of the gasoline locomotive over the steam locomotive deserve especial emphasis. The first is that the engine driver on a gasoline locomotive has a much better opportunity to see what he is doing and how his train is being handled, since he looks directly out upon it from the cab and does not have his vision interfered with by the boiler or the tender or clouds of escaping steam, as happens very often in steam switching service. Again, in the operation of these locomotives, the fireman is, of course, dispensed with, making a large saving in running expenses. Still further, as escaping smoke and steam and the noise of the exhaust are eliminated, the machines can be operated in places where

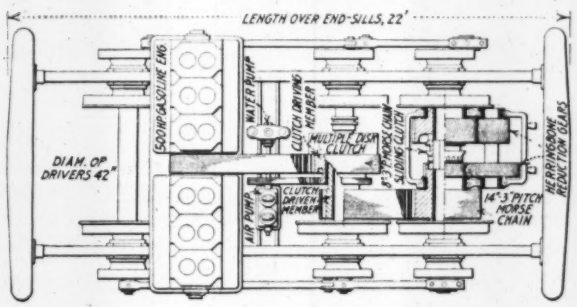


FIG. 2. PLAN OF MACHINERY FOR GASOLINE-ENGINE-DRIVEN SWITCHING LOCOMOTIVE OF 500 HP.
(Designed by the McKeen Motor Car Co.)

a steam switching locomotive would give rise to perpetual complaint and threats of damage suits or injunctions.

Turning now to the gasoline locomotives actually on the market, we illustrate in Fig. 1 a 300-hp. gasoline locomotive recently built by the McKeen Motor Car Co., of Omaha, Neb., for the Motley County Ry. of Texas. This machine is now in use in freight and switching service between Roaring Springs Junction on the Quanah, Acme & Pacific Ry. and the town of Matador, Tex. As showing its capacity for road service, it is interesting to note that the machine made the trip from Omaha to its destination under its own power.

The locomotive has a total weight of 66,000 lb., of which 46,000 lb. is on the four driving wheels. The gasoline engine used is a six-cylinder machine, the cylinders being 11 in. in diameter and 15 in. stroke, corresponding in general design to the engines used on the McKeen motor cars, built by the same company.

The general arrangement of the machinery is shown in Fig. 2, which, however, is a plan of the machinery used on a locomotive of 500 hp., for which the McKeen Co. has made the designs but which has not yet been built.

From the center of the crankshaft a Morse chain drive rotates a sleeve which turns freely on the rear driving axle. A multiple-disk friction clutch on this axle permits the sleeve to rotate freely when the clutch is disengaged, or puts it in connection with a sprocket driving a Morse chain passing over a sprocket on the front driving axle. A clutch on the latter axle enables the engine to either deliver power direct to the driving wheels or to drive them through the medium of a series of speed-reducing gears, the latter being used, of course, when the locomotive is starting a train or hauling heavy loads. When in low gear, and running at a speed of six miles an hour, the 300-hp. locomotive is designed to produce a drawbar pull of 12,000 lb. The clutch is operated by compressed air, supplied by a two-cylinder 5-in. air compressor driven from the engine crankshaft, which also furnishes air supply for the brakes. There is also provided an auxiliary air compressor in the cab for emergency service.

A notable feature is that herringbone gears are used, giving great strength to withstand the shocks of service. The water circulated from the cylinder jackets is cooled by the tubular radiator seen on the roof of the engine cab, which has 3600 sq.ft. of cooling surface. On high gear the locomotive can make 16 miles per hour with the engine running at 350 r.p.m.

Mr. McKeen has also designed a gasoline locomotive of 500 hp., the general construction being similar to that of the 300-hp. locomotive herewith illustrated, all parts being enlarged in proportion to their increased work.

While this 500-hp. locomotive has not yet been constructed, a gasoline-electric locomotive of smaller nominal power but greater hauling capacity has been built by the General Electric Co. for the St. Paul, Rochester & Dubuque Electric Traction Co. It is in service between Minneapolis and Northfield, Minn. This locomotive has two 8-cylinder gasoline engines of the type used on the General Electric motor cars. Each of these units drives an electric generator supplying current to electric motors mounted on each of the four axles of the locomotive The total weight of the machine is 114,000 lb. and it is claimed to have a maximum draw-bar pull of 30,000 lb. The engines are rated at about 400 hp. The cost of this machine, with its necessarily expensive electrical equipment, is doubtless considerably greater than that of the McKeen machine, with its straight mechanical gear.

Gasoline switching locomotives built by the Baldwin Locomotive Works, and having speed-reducing gears similar in general design to those used on automobiles, were described by A. H. Ehle, manager of the gasoline-locomotive department of the company, in a paper read before the Society of Automobile Engineers, at its annual convention in New York on Jan. 7. The heaviest of these locomotives is a machine weighing 18 tons, capable of developing a drawbar pull on low gear of 6000 lb.

According to Mr. Ehle, one of these machines built for the Kellogg Toasted Corn Flakes Co., in a recent test exceeded its guaranteed drawbar pull by about 50%, when operating on either gasoline or kerosene. The fuel consumption was found to be about 4¼ gal. per hr. when hauling a train of 80 tons back and forth over a track having level stretches and grades as high as 3%, the engine running on the low gear.

These illustrations are sufficient to show the rapid development of the gasoline switching locomotive for industrial purposes and the possibilities of its use for lighter switching service in railway yards.

Wilson Ave. Water Tunnel at Chicago

SYNOPSIS—The water supply of Chicago is to be increased by a new intake tunnel extending three miles under the lake to an intake crib and five miles under the city to a pumping station of 100,000,000 gal. daily capacity. The tunnel will be of horseshoe section, 13 ft. wide and 12 ft. high, and will be at such a depth as to keep within the solid rock. The tunnel will be built by the city, on the day-labor plan. The intake crib and shaft and the tunnel connection will be built by contract. Work is already in progress on the tunnel.

The water supply of the city of Chicago being taken from Lake Michigan, the earlier pumping stations were located near the shore, and with the rapid growth of the city, many of the pumping or force mains of the distribution system have been extended until they are of uneconomical length and of insufficient capacity toward their extremities. The policy of the Department of Public Works is to improve these conditions by dividing the city into districts, each with a pumping station located approximately at its center. These stations have a normal capacity of 100,000,000 gal. daily, and in order to supply this flow economically they are served by gravity tunnels forming extensions of the lake intake tunnels.

One of the latest of these district water-supply systems is that known as the Southwest system, the tunnels for which were completed in 1911. This has an intake crib and shaft 2.1 miles from shore, with a tunnel to a shore shaft and land tunnels under 73rd St. and State St. to serve three pumping stations.* A second system, which has been commenced recently for the benefit of the northwest section of the city, comprises a 3-mile lake intake tunnel and five miles of land tunnel under Wilson Ave. to a pumping station at Lamon Ave. (Mayfair). The plan and profile of this are shown in Fig. 1.

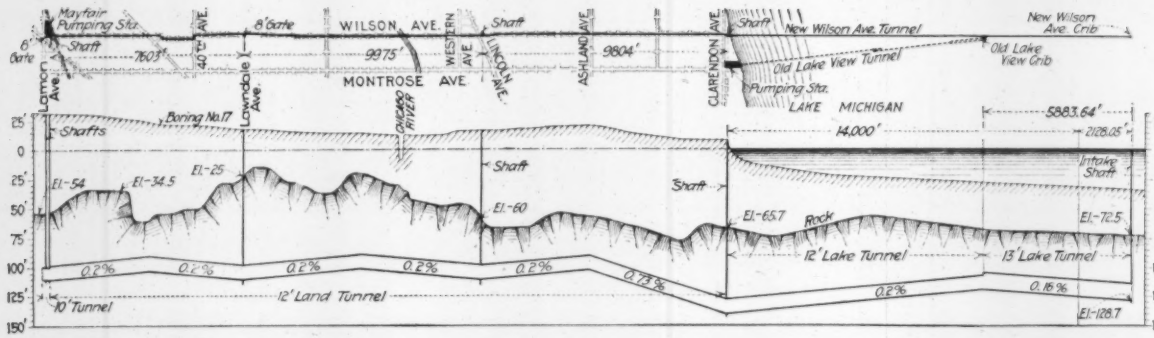


FIG. 1. PLAN AND PROFILE OF THE 8-MILE WILSON AVE. TUNNEL FOR INCREASING THE WATER SUPPLY OF CHICAGO

TUNNELS

In many of the older lake intake tunnels difficulties were encountered due to the bad character of the ground, most of these tunnels being in the material overlying the rock. In several cases deviations in line and grade were necessary to avoid or by-pass these troublesome points,† and this was the case in the old tunnel shown in Fig. 1. The 73rd St. tunnel mentioned above was located at a much deeper level in order to insure solid rock throughout, its invert being 153 and 137 ft. below datum at the intake shaft and shore shaft respectively. The Wilson Ave. tunnel is located at slightly less depth, its invert being at 128 ft. and 140 ft. at lake and shore shafts respectively.

In order to determine the depth and character of bedrock, the usual auger borings were replaced by a number of bore-holes put down with a diamond drill (ENGINEERING NEWS, Dec. 12, 1912). These showed solid rock at depths of 57 to 75 ft. Above this were beds of clay and gravel of varying character, the former including soft clay, liver or "quaking" clay, stiff clay and hardpan. Similar borings on the line of the land tunnel showed rock at depths varying from 14 to 78 ft. These borings cost about $15.08 and $1.51 per lin.ft. for the lake and land tunnels respectively.

The intake tunnel will have vertical sides, a semicircular roof and an invert floor. It will be lined with concrete having a normal thickness of 12 in. and a minimum thickness of 10 in. For about 5883 ft. from the intake shaft the tunnel section is 12 ft. 3 in. wide and 13 ft. high, and for the remainder of its length it is 11 ft. 4 in. wide and 12 ft. high. The shore tunnel is of the latter section throughout. These sections are shown in Fig. 2, and their respective properties are given in the accompanying table.

PROPERTIES OF SECTIONS OF THE WILSON AVE. WATER TUNNEL AT CHICAGO

	13-Ft. Section from Intake Shaft	12-Ft. Section To Shore Shaft	12-Ft. Section Shore Shaft to Lincoln Ave. Shaft	12-Ft. Section Lincoln Ave. to Mayfair Shaft (Pump Station)
Length, ft.	5883	10,246	9804	17,578
Area, sq.ft.	139.29	118.78	....	....
Perimeter, ft.	43.52	40.16	....	....
Hyd. radius, ft.	3.20	2.96	....	....
Velocity, ft. per sec.	3.89	3.59	3.26	2.60
Capacity, mill. gal. per 24 hr.	350	275	250	200
Loss of head in ft. per 1000 ft.*	0.284	0.265	0.220	0.140
Concrete cu.yd. per ft. in 12-in. ring	1.74	1.61	....	....
Rock excav. cu.yd. per ft. (to neat line)	6.90	6.01	....	....

*Total loss of head in main tunnel (intake shaft to pump shaft), 9.01 ft.

*Engineering News, Nov. 14, 1907.

†Engineering News, Oct. 18 and Nov. 8, 1900.

From the foot of the intake shaft there is an ascending grade of 0.16% for 5883 ft., followed by a descending grade of 0.2% to the shore shaft. Thus the tunnel will drain to both shafts when it becomes necessary to pump it out for inspection. From the shore shaft is a grade of 0.73% to the higher level of the land tunnel, which tunnel has grades of 0.2% descending in both directions to the shafts. The depth at the pump shaft will be about 141 ft., or 110 ft. below datum (water level).

It is the practice of the city to build all its tunnels under the streets in order to avoid trouble from building foundations, etc. Irregularities in the street layout necessitate bends in the line of the tunnel at three points.

Tunnel Connections

Beyond the pump shaft at the Mayfair end of the land tunnel there is to be a short stub tunnel of 10-ft. horseshoe section (Fig. 2). At Lincoln and Lawndale Aves. there will be a lateral branch of 8-ft. circular section. These are to afford connections with other water tunnels.

In addition to its main purpose of supplying the new Mayfair pumping station, the Wilson Ave. tunnel will afford an auxiliary supply to the present Lake View pumping station, about three blocks south of the shore shaft (Fig. 1). This latter is served by a 6-ft. intake tunnel to the Lake View crib (2 miles), but this will be insufficient for the new pumping plant with a total capacity of 100,000,000 gal. daily. An 8-ft. circular tunnel has been built, therefore, to connect the shore shaft of the new tunnel with the Lake View pumping station, about 1400 ft. This was built in 1913 by the day-labor system under the Department of Public Works, at a cost of $23 per ft. of completed tunnel.

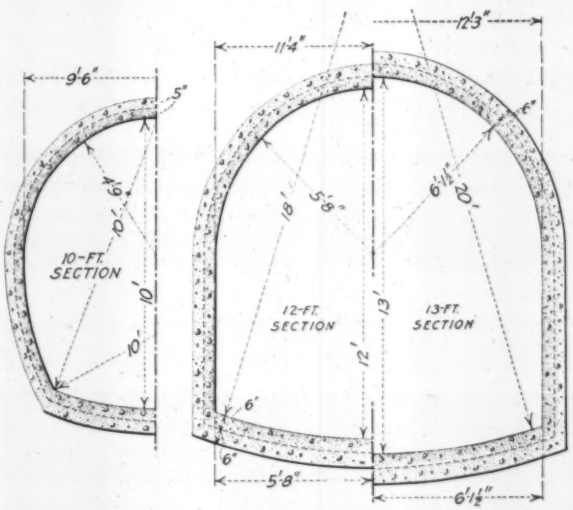


Fig. 2. Cross-Sections of the Wilson Ave. Tunnel, Chicago

Besides this a connection will be made between the new 12-ft. intake tunnel and the intake shaft of the old 6-ft. intake tunnel, so that the Lake View or two-mile intake crib may be abandoned and both tunnels supplied with water from the new three-mile crib.

Shafts

The intake shaft at the new crib is shown in Fig. 4. It will be 14 ft. diameter inside the concrete lining, which will be 18 in. thick in soft material and 12 in. thick in rock. It will be extended up within the well of the crib by means of a cylinder of cast-iron sections, 6 ft. high, put together with interior flanges and bolts, all covered with an 8-in. concrete lining. This cylinder will be extended far enough beneath the floor of the crib to reach stable material.

During the construction of the shaft and tunnel, this cylinder will form part of the working shaft, and its top

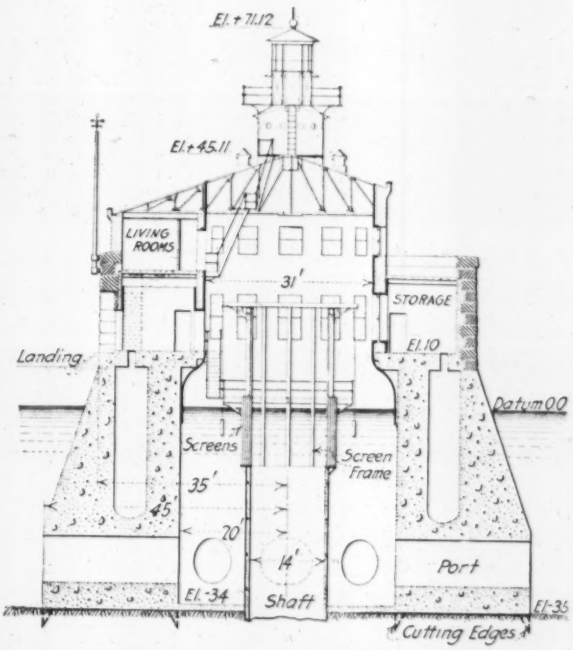


Fig. 3. Typical Cross-Section of the Intake Crib for the New Wilson Ave. Water Tunnel, Chicago

will be about 5 ft. above water level. When the tunnel is completed, the upper sections will be removed, to a depth of 10 ft. below water level, and the cylinder surmounted by the columns and framework for a vertical cylindrical screen structure.

The shore shaft, at Wilson Ave. and Clarendon Ave., is 12 ft. inside diameter, with the bottom 140 ft. below datum. For use as a construction shaft it has a 3-ft. sump, but this will be filled after the completion of the tunnel.

This shaft was sunk from a timbered pit 18 ft. square and about 8 ft. deep. To pass through strata of loose and water-bearing sand a steel shell 13½ ft. diameter was sunk until it had entered well into the blue clay with its lower edge 28 ft. below datum. Here the excavation was enlarged and the sinking continued in open cuts with lifts of about 4 ft., each lift being lined with vertical 3-in. tongued-and-grooved lagging supported by two interior ring braces 4x1 in. This method was followed until solid rock was reached (about 65 ft.), and drilling and blasting were then employed. The concrete lining is 12 in. thick in rock, 15 in. inside the timber lagging and 9 in. inside the steel shell. The shaft cost $106.70 per ft. in depth.

Intake Crib

The intake crib will be located three miles from shore, in water 35 ft. deep, with its intake ports about 4½ ft. above the lake bottom and their tops 22½ ft. below the surface. Its port area is sufficient to give the crib a

flow capacity of 350,000,000 gal. per 24 hr., with a velocity of only 1.02 ft. per sec. in the ports. Fig. 3 is a typical cross-section of the crib and superstructure.

The crib will consist essentially of two concentric steel cylindrical shells, connected by radial partitions to divide the annular space into sixteen compartments or pockets, which will be filled with concrete when the crib has been sunk in place. The shells are 42 ft. deep, with cutting edges as shown. The inner one will be 40 ft. diameter. The outer one will be 90 ft. diameter for about 14 ft.

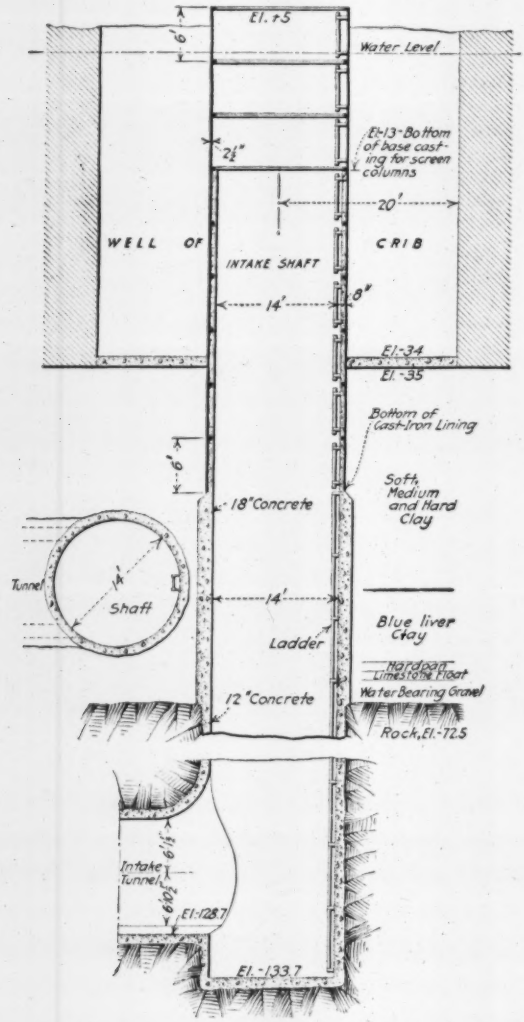


FIG. 4. SECTION OF THE INTAKE SHAFT OF THE WILSON AVE. TUNNEL, CHICAGO

from the bottom and then tapering to 70 ft. at the top. This gives a conical or inclined face to break the force of the waves and of ice pressure.

There will be eight intake ports 8x10 ft., extending through the concrete and leading from the lake to the interior well of the crib. Within the well will be the top of the intake shaft, with screens, as described above.

Upon the crib substructure will be a circular wall of granite masonry forming the outer side of an annular structure surrounding the well. Above this, but not extending entirely around the crib, will be an upper story with living quarters for the attendants maintained at the crib. A steel truss roof will extend over the building and well, and will be surmounted by a tower carrying lights as a warning to vessels. On the most sheltered side will be a concrete landing, with steps. This will be built on wood piles and surrounded by steel sheetpiling.

There is no bottom to the compartments, but sufficient buoyancy for floating the crib into place will be furnished by means of the ports (closed temporarily with bulkheads) and by vertical flotation cylinders, one in each compartment. The crib will be sunk by admitting water to these closed spaces. The compartments will be pumped out one by one and filled with concrete laid in the dry. If the cutting edges should fail to seal any one of the pockets, clay will be deposited on the outside (and in adjacent pockets if necessary) to enable the pocket to be pumped out. The bottom of the lake will be dredged to give a level surface for the crib, so that it will land in a vertical position.

The conditions of stability and loading of the crib are as follows:

1. Steel Crib Riveted Complete and Floating. Total weight, 656 tons; center of gravity 19 ft. above the bottom of shell:

Volume required to float	21,000 cu.ft.
Volume of 8 ports	13,230 cu.ft.
Volume of 8 cyls. 10 ft. diam.	9,080 cu.ft.
Volume of 8 cyls. 6 ft. diam. to datum	4,250 cu.ft.
Draft, mean	17.5 ft.
Force due to 30-lb. wind	48,000 lb.
Wind moment	1,056,000 ft.lb
Draft with wind (max.)	21 ft.

2. Sinking the Steel Crib. The volume of water required is 8000 cu.ft., and the approximate time is calculated at 5 min. The load will be 1785 lb. per lin.ft. of cutting edge and 297 lb. per sq.ft. on the contact area of the 3-ft. fins which stiffen the cutting edge.

3. Placing the Concrete. The steel shells are designed to withstand the hydrostatic pressure when water is pumped out of the compartments so that concrete may be placed in the dry.

4. Substructure and Superstructure Completed (resting on the bottom in 35 ft. of water). The total weight on the foundation will be 14,000 tons or 5480 lb. per sq.ft. of bottom. The overturning moment due to a 30-lb. wind and ice at 40,000 lb. per lin.ft. is 101,360,000 ft.-lb. This gives a pressure of 1480 lb. per ft. on the toe, or a maximum pressure on the bottom of 6960 lb. per sq.ft.

CONSTRUCTION WORK

Bids for the land tunnel were opened June 25, 1914, but were rejected as being too high. The city engineer considered that the work could be done at less expense by day labor, and on July 2, 1914, the city council granted permission to the Department of Public Works to carry out the construction on this system. Up to December, 1914, the shore shaft had been completed, and work commenced on the three other shafts. About 580 ft. of tunnel had been driven west from the shore shaft. A notable feature is that the plant is electrically operated, including the hoists, compressors, pumps, concrete mixers, etc.

As the construction of the lake intake crib and shaft will involve special methods and equipment it is proposed to have this part of the work done by contract, and bids were opened Dec. 2, 1914. The contract for this intake crib and shaft and for 2128 ft. of tunnel run-

ning west from the shaft was let to the Fitzsimons & Connell Dredge & Dock Co., of Chicago, at $403,179. This amount is distributed as follows: Crib, $159,070; superstructure of crib, $42,000; landing at crib, $17,818; intake shaft, $20,435 ft.; tunnel (2128 ft.), $163,856.

The Department of Public Works proposes to drive the lake tunnel by day labor for 14,000 ft. east from the shore shaft, to meet the contract section. This day-labor work had advanced about 2000 ft. by January, 1915. It may be continued beyond the 14,000 ft. if the contract section is not then completed. It is expected to have the land tunnel completed by 1918, and the intake crib and lake tunnel completed and ready for service about the same time. The total cost for the tunnels and shafts is estimated at $4,550,870.

The design and execution of the work are under the direction of John Ericson, City Engineer, and H. S. Baker, Assistant City Engineer. The construction work on the entire tunnel is under the direction of H. W. Clausen, Engineer of Water-Works Construction.

Street Contours at Intersections and Crosswalks*

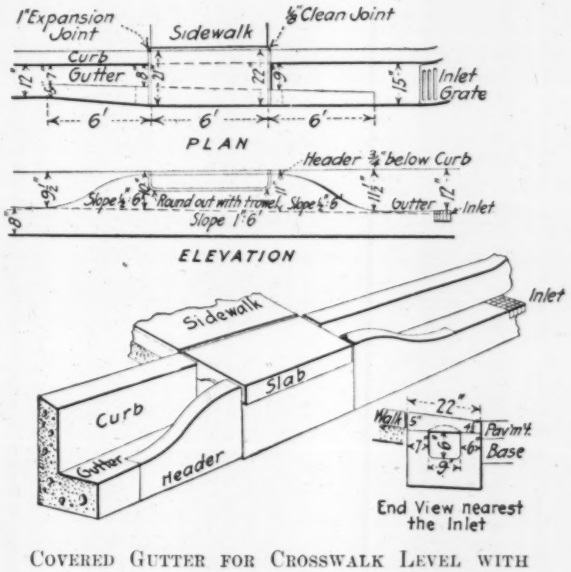
In Chicago street pavement construction it is the general policy to eliminate the step from the curb to the street level in residence and retail business streets, both at street intersections and at alley returns where pedestrians cross. This is accomplished by paving the street or alley flush with the curb in line with the continuation of the usual 6-ft. sidewalk. To drain the intersection, the catchbasin inlet at the curb corner is set 4 to 6 in. below the top of the curb. From the line of the outer edge of the sidewalk the pavement slopes on a straight line to the nearest sewer inlet.

In residence districts, where there are no sewers in the side frontage streets, and the catchbasins are already built in the customary manner (at the curb corners and in the middle of the standard 600-ft. block), it is often advisable to save expense by a method shown in the accompanying drawing. A raised header is built in the combined curb and gutter, with its top 1 in. below the curb and recessed in the middle of its top for a 5-in. slab 6 ft. wide, which is an extension of the adjacent 6-ft. sidewalk. The header is 6 in. thick and is so built as to make the width of the gutter between the curb and header wider at the end nearest the inlet. The gutter is built with an increased slope under the slab. The purpose of this arrangement is to render the gutters self-cleaning, especially under the slab. The header is connected to the surface of the gutter by a reverse curve for a distance of 6 ft. on each side of the extended sidewalk, or slab cover, making the header 18 ft. long.

This construction has been in use for four years, and has proved satisfactory. It eliminates the step at the crossing, covers the usual dangerous opening where a false gutter is employed, affords a dry and clean crosswalk at all times, and prevents breaking of the curb through expansion of the sidewalk. The slab cover is not slippery and cannot be displaced. H. J. Fixmer has patented this construction and has given the City of Chicago the right to use it.

At alley returns, the pavement is brought flush with the top of the curbs for the full width of the sidewalk. At the end of the return, or street line, the center of the alley is dished at the rate of 1 in. per 5 ft. of width and this point is connected on a straight line to the gutter line of the street proper, affording good drainage and keeping the crossing dry and clean.

The ideal solution for a business street is to bring the pavement flush with the walk of the street at the returns for the cross streets, run the pavement surface at the gutter line to the curb corner at a depth of about 4 to 6 in. below the top of the curb, thence around the curb corner to the catchbasin inlet, located about 70 ft.



COVERED GUTTER FOR CROSSWALK LEVEL WITH SIDEWALK, AS USED IN CHICAGO

(depending on the grade) from the curb corner, at a depth of about 8 to 10 in. below the top of the curb. This eliminates any step for the main lines of pedestrian traffic along the business street, but does present a 6-in. step to those crossing that street. This tends to cause pedestrians to hesitate before crossing a busy street, and affords a smooth full width of roadway for the vehicle traffic along the direction of the busy street.

*From a description by H. J. Fixmer, Division Engineer of the Board of Local Improvements, at the recent annual meeting of the Illinois Society of Engineers and Surveyors, at Springfield, Ill.

A Neglected Source of Municipal Income—If the practice of other cities is similar to that of Philadelphia, there is being neglected a just and legitimate method of gaining an income which might well be used for street-pavement repair and maintenance. Wm. H. Connell, in his recent annual report on the Philadelphia Bureau of Highways, brings out that during 1914 there were 7045 permits issued for placing building material on streets, from which was derived a revenue of $1,761.25. The ordinance governing this specifies a charge of only 25c. a month for permission to occupy from 80 to 100 ft. of roadway for a width of 9 ft., or about ¼c. a month per lin.ft., for the storage of building material on the highways. If this charge, for example, were increased to $5 per month, which would be $60 a year and would seem to be a very reasonable charge for occupancy of the highways in connection with a building operation and a very small proportion of the cost of the work, the revenue from this source alone would be increased about $40,000, basing the calculation on an average of 9000 permits each year.

Temporary Water Aërator at the Kensico Reservoir

By Wilson Fitch Smith*

The New Rye aërator, built in connection with the temporary reservoir at Kensico, a part of the Catskill water-supply for New York, was briefly described in Engineering News of Dec. 31, 1914.

The aërator is designed to pass the maximum reservoir draft of 18,000,000 gal. a day. It consists of 44 special and 20 plain nozzles, set in a 24-in. nozzle-feed pipe, shaped in plan like a slightly flattened ring with a cross-connection on the shortest, or 20-ft., diameter. This pipe is embedded in the floor of a shallow concrete basin, nearly circular in plan, with a diameter of about 100 ft. Two short runs of 24-in. pipe connect the aërator with the 36-in. supply main from the reservoir. One supplies the nozzle-feed pipe; the other conveys the aërated water from a downtake in the basin back to the supply main. When the aërator is in use the valves on the influent and effluent connections (Fig. 2) are opened and the valve on the supply main, between the points of connection, is closed. All the piping is of standard cast iron.

*Division Engineer, Kensico Division, Board of Water-Supply, City of New York, Valhalla, N. Y.

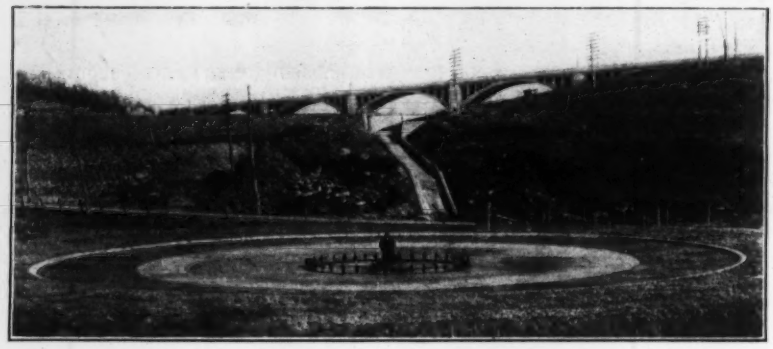


Fig. 1. Temporary Aërator at the Kensico Reservoir, Catskill Water-Supply

The 44 special nozzles were designed after considerable experimentation. They consist of a 4-in. cylindrical body (see detail in Fig. 2), on the inside of which are three radial spiral-shaped vanes, which project nearly to the center of the nozzle. Above the tops of the vanes the body of the nozzle contracts in a conical shape to a short orifice, 1⅜ in. in diameter. The nozzles are of cast bronze, flange connected to the cast-iron bases. This form, owing to the decided rotary motion which it imparts to the jet, is

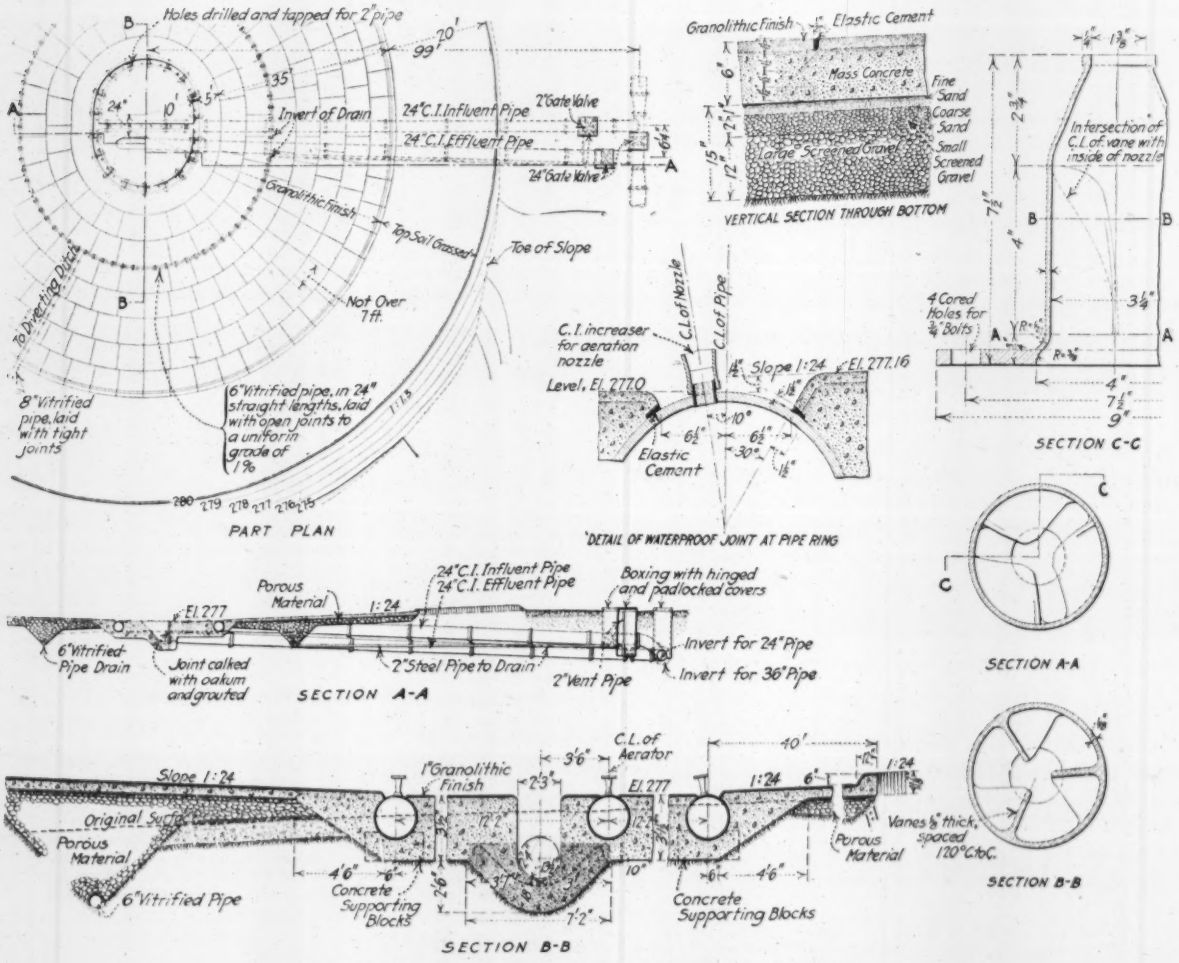


Fig. 2. Details of Aërator at Kensico Reservoir

most efficient in breaking the latter into a fine spray a short distance above the nozzle, resulting in the exposure of the individual drops to the air for several seconds. This exposure is sufficient to liberate entrained gases and to cause the oxidization of the organic matter carried by the water, thus removing the sources of tastes and odors.

As it was found desirable to operate the aërator at a smaller head than it was designed for, the capacity was increased by the addition of the 20 straight nozzles.

Owing to the large amount of entrained air carried by the water after aëration, the capacity of the effluent pipe was largely reduced. This was readily overcome by placing a number of 2-in. vent pipes on the effluent pipe and on the supply main, which liberate the air as it collects at the top of the pipe. This suggests that a short stretch of open channel or air remover is an essential feature in the design of an aërator.

Automobiles versus Horses in Road-Work Supervision

By E. W. James*

In the spring of 1914 the United States Office of Public Roads undertook the supervision of maintenance over a long mileage of road from Washington, D. C., to Atlanta, Ga. Three engineers were assigned from the Division of Maintenance to three sections of the road, known for administrative purposes as the Northern, Central and Southern sections. These sections were respectively 404, 272.1 and 303.3 miles long.

Not all the road came under government supervision, because such relation depended on voluntary action by the local road authorities, and some refused to come into the plan. But the work was so nearly continuous and so widely distributed over the sections that the supervising engineers had to cover their entire sections.

Small motor cars (costing about $575 delivered) for use of the supervising engineers were furnished by the American Highway Association, through Leonard Tufts, proprietor of Pinehurst, N. C. The association cooperated with the Office of Public Roads by leasing these cars at a nominal rate for one year or longer at the option of the lessor. Exact records were kept from the time the cars were taken over by the supervising engineers, and these cost records are compared with similar work done with livery rigs.

The accompanying table gives the itemized cost of operating the three cars. These figures do not include depreciation or interest on capital investment, and to compare them with livery charges for similar work such charges should be added. As noted the cars cost approximately $575 each delivered at the point where the supervising engineers took charge. The interest per car for seven months at 6% is $18.96. Assuming the life of the car as equivalent to 30,000 miles run, which is very conservative, the depreciation chargeable at the end of the present record is respectively $119.02, $178.40 and $166.98. Adding these charges to the total cost of running the cars, we have:

Car	No. 1	No. 2	No. 3
Actual operation	$273.58	$353.17	$318.12
Depreciation	119.02	178.40	166.98
Interest	18.96	18.96	18.96
Totals	$411.56	$550.53	$504.06

Average $519.30 for a period of seven months.

*Chief of Maintenance, U. S. Office of Public Roads, Washington, D. C.

In comparison with this cost we have the following actual figures for livery used by supervising engineers on road work during an equivalent period, which in large part was the same period:

State	Charges	Length of Project in Miles
Alabama	$554.17	30
Iowa	437.44	30
Oregon	649.99	50
Mississippi	296.04*	24

* Some transportation was furnished by contractor that does not show in these figures.

These figures are for seven months in each case, and the common average of the livery charges for the four projects is $484.41. If the figures for Mississippi are omitted, the average is $547.20.

The mileage traveled by horse vehicles was rarely over 25 miles in any day and often was much less, and the charges are not for continuous daily service. In the case of the automobiles, the use has been almost constant, and the average daily run was about 35 miles for 30 days per month.

Had livery been used on the Southern section of the Washington-Atlanta Road between Augusta and Atlanta, via Athens, the time required to make one complete inspection would have been about one week. The same inspection could be made in three days with the automobile.

This increase in efficiency and saving in time cannot be stated fairly in dollars and cents, but represents a great saving. Furthermore, the mileage in charge of one man using automobile can be about twice that of a man using livery, without decreasing the number of visits or closeness of supervision.

To reduce these two sets of costs to a comparable basis, it is necessary to establish the cost of inspection per mile of road covered. The distances under actual supervision in the three sections of the Washington-Atlanta Highway are 231.1, 208.1 and 242.6 miles respectively, making a total of 681.1 miles. The cost per mile of transportation for supervision during seven months was, therefore, $0.762 when the automobile was used. Similarly, including the four localities cited, the cost per mile for seven months when livery was used was $3.61½.

TABLE SHOWING ITEMIZED COST OF OPERATING SMALL MOTOR CARS ON ROAD INSPECTION WORK

Item	Car No. 1	Car No. 2	Car No. 3	Common Average
Average cost of gasoline per gal.	20.01c.	18.96c.	16.77c.	18.58c.
Miles run per gal. of gasoline	17.52	17.45	19.73	18.23
Miles run per pound of grease	282.28	340.58	458.53	360.46
Miles run per quart of oil	56.97	81.65	86.26	74.96
Gasoline cost per mile	1.14c.	1.09c.	0.85c.	1.03c.
Grease cost per mile	0.05	0.05	0.03	0.04
Oil cost per mile	0.24	0.17	0.17	0.19
Tire casings per mile	0.15	0.90	0.48	0.84
Other tire costs per mile	0.17	0.07	0.20	0.15
Storage charges per mile	0.89	0.28	0.64	0.60
Miscellaneous costs per mile	0.76	*1.23	*1.28	1.09
Total cost per mile	4.40c.	3.79c.	3.65c.	3.94c.

* Included general overhauling at end of November.

Railway Improvements in India are being carried out on a number of lines in order to increase the traffic capacity of the main through routes to the ports of Calcutta, Bombay and Karachi. The government report for 1913-1914 states that owing to the congestion experienced on different lines there has been a general public demand for the provision of additional rolling stock, but the Railway Board and the individual railway administrations have felt that this would be almost useless unless steps are taken to improve the capacity of the lines on which the new equipment is to operate. A considerable amount of money has been devoted, therefore, not only to the provision of additional rolling stock but also to heavier track construction, additional tracks, enlargement of freight yards, more adequate water-supply, and increase in shops and shop facilities. Further expenditures of this kind will have to continue for some years.

Sewer Tunnel at San Francisco

An outfall sewer now being built at San Francisco, Calif., has its main portion in a rock tunnel 4380 ft. long, extending from Cabrillo St. under Sutro Heights to the outlet at the sea front. The tunnel is of horseshoe section, 9x11 ft., lined with concrete. The construction is handled from two adits 3116 ft. apart, one at 90° and the other at 20° with the center line. The work is carried on from both ends in three 8-hr. shifts, two for the excavation and the third for the concreting.

The south heading is in hard blue shale, and the north heading passed through blue sandstone before entering the same shale. The drilling in the south heading is done by two hand drills. From 25 to 30 lb. of 40% dynamite are used per round. In the north heading, where the rock is harder, two piston drills mounted on columns have drilled an average of 18 holes 6 ft. deep, requiring from 50 to 70 lb. of dynamite per round. The timbering consists of a two-post, five-segment arch of 8x8-in. timbers, spaced about 5 ft. c. to c. Lagging is used only in bad spots. Mules haul out the muck in ¾-yd. narrow-gage cars.

The concreting is done by the MacMichael pneumatic system, the compressed-air plant for the drills serving this work also. The concrete is mixed in ¼-yd. batches. The conveyor pipe is 1300 ft. long, but will be extended to about 2000 ft. At the present distance the average is about 32 batches of concrete per hr., but at 600-ft. distance the average was about 60 batches. On the end of the pipe is 4 ft. of rubber hose. The concrete is 1:2¼:5½, made with 2½-in. gravel.

The excavation force in each heading consists of a foreman, four miners and six shovelers. For concreting, there are two men at the mixer and two at the forms, electric signals being used to regulate the operations. M. M. O'Shaughnessy is City Engineer of San Francisco. The general contractors are R. C. Storie & Co., of San Francisco. The work was commenced in August, 1914, and will be completed this year.

Specifications for the Application of Resurfacing Asphalt Binder

The following specifications have been prepared by the engineers of the Standard Oil Co. of New Jersey, for the annual spring treatment of bituminous road surfaces by means of a motor-truck pressure distributor. The table gives the theoretical quantities of oil required:

1. Before any bituminous material is applied the surface of the road shall be thoroughly swept, either by means of horse-drawn brooms and hand-brooms, or by hand-brooms only, until all loose dust and dirt are removed. If considered necessary the roadway shall be scraped of all accumulations of dirt, until a clean, coarse surface is obtained. This material shall be hauled away and must not be used to cover the asphalt binder. If so directed, and in order to more effectively clean the surface, the road shall be watered before applying the binder. Holes, ruts and depressions shall be filled with a mixture of stone and asphalt thoroughly compacted.

2. The asphalt binder shall be applied under pressure at a temperature of about 250° F. with an approved motor-truck distributor, at a rate of ... gal. per sq.yd. (The quantity applied will depend on the character of surface and will vary from ½ gal. to 0.7 gal. per sq.yd. For average conditions, 0.6 gal. per sq.yd. will be found satisfactory.) If a motor-truck distributor is not available application by hand may be resorted to, care being taken that a uniform quantity per square yard is secured.

3. After the application of the asphalt enough screenings, pea-stone or grit, shall be spread over the road to cover the binder and provide a suitable wearing surface. The quantity in each case will depend upon the character of the material and the rate of application of the asphalt, and shall be determined by the engineer. In general, approximately 35 lb. of cover per sq.yd. of surface should be provided.

4. A sufficient quantity of covering shall first be spread over the hot asphalt binder as quickly as possible, after which additional quantities shall be added to give the surface the required thickness.

5. Where the traffic is confined exclusively to motor vehicles coarse sand may be used for the cover, but where there is considerable horse-drawn traffic, the use of sharp, clean screenings is recommended. When considered advisable by the engineer the road should be thoroughly rolled with a steam roller to more thoroughly consolidate the sand or screenings.

TABLE OF THEORETICAL AMOUNTS OF OIL REQUIRED FOR ROAD SURFACE TREATMENT

Width, ft.	Sq. yd.	Quantity in Gallons per sq.yd. for Varying Applications									
		0.5	0.6	0.7	0.8	0.9	1.00	1.25	1.50	1.75	2.00
12	7,040	3520	4224	4928	5632	6,336	7,040	8,800	10,560	12,320	14,080
14	8,213	4107	4928	5749	6570	7,392	8,213	10,266	12,320	14,373	16,426
15	8,800	4400	5280	6160	7040	7,920	8,800	11,000	13,200	15,400	17,600
16	9,386	4693	5632	6570	7509	8,447	9,386	11,733	14,079	16,426	18,772
18	10,560	5280	6336	7392	8448	9,504	10,560	13,200	15,840	18,480	21,120
20	11,733	5867	7040	8213	9386	10,560	11,733	14,666	17,600	20,533	23,466

Imhoff Sewage Tank at Manchester, England

By Oswald J. Wilkinson*

The first Emscher or Imhoff sewage tank to be constructed in Great Britain was placed in operation by the city of Manchester, England, in the latter part of 1914. The installation has a daily capacity of 750,000 Imp. or 900,000 U. S. gal. There is a grit chamber, an 18-in. Venturi meter, a coarse screen, an Imhoff tank in two

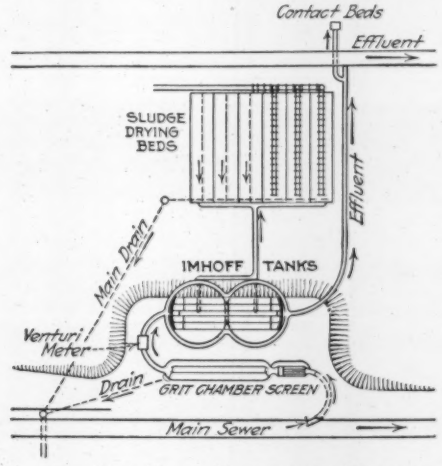


Fig. 1. Imhoff Tank Layout at Manchester, England

compartments, each 26 ft. in diameter, and sludge-drying beds 58x78 ft. in surface plan, giving an area of 4408 sq.ft. Provision has been made for measuring the gases given off by the tank. Fig. 1 shows the general layout and Fig. 2 the details of the plant.

Although two hundred Imhoff tanks had been built in Germany up to 1913 no public authority in England had investigated the advantages claimed for this type of tank. The city of Manchester, recognizing that the results obtained from Imhoff tanks abroad were exceptional, decided to make a small installation at its Withington works, to treat a strictly domestic sewage works. The writer,

*Consulting Civil Engineer, 196 Deansgate, Manchester, England.

acting as consulting engineer to the city, was instructed to design and carry out the works.

The works will be operated under the close supervision of chemists and bacteriologists in order that the city may comply with the request of the Local Government Board that careful data be compiled for use in future installations.

A Contract May Be Voided under the Strike-and-Fire Clause, according to a recent decision of the Supreme Court of Massachusetts. A lumber company which contracted to supply a certain amount of maple flooring at a stipulated price had a fire in the mill that was to turn out the flooring, before any had been manufactured or delivered. It thereupon notified the purchaser that it would be unable to fulfill the contract. The purchaser sued, as the lumber company, presumably, was not unable to fulfill its part of the contract, but

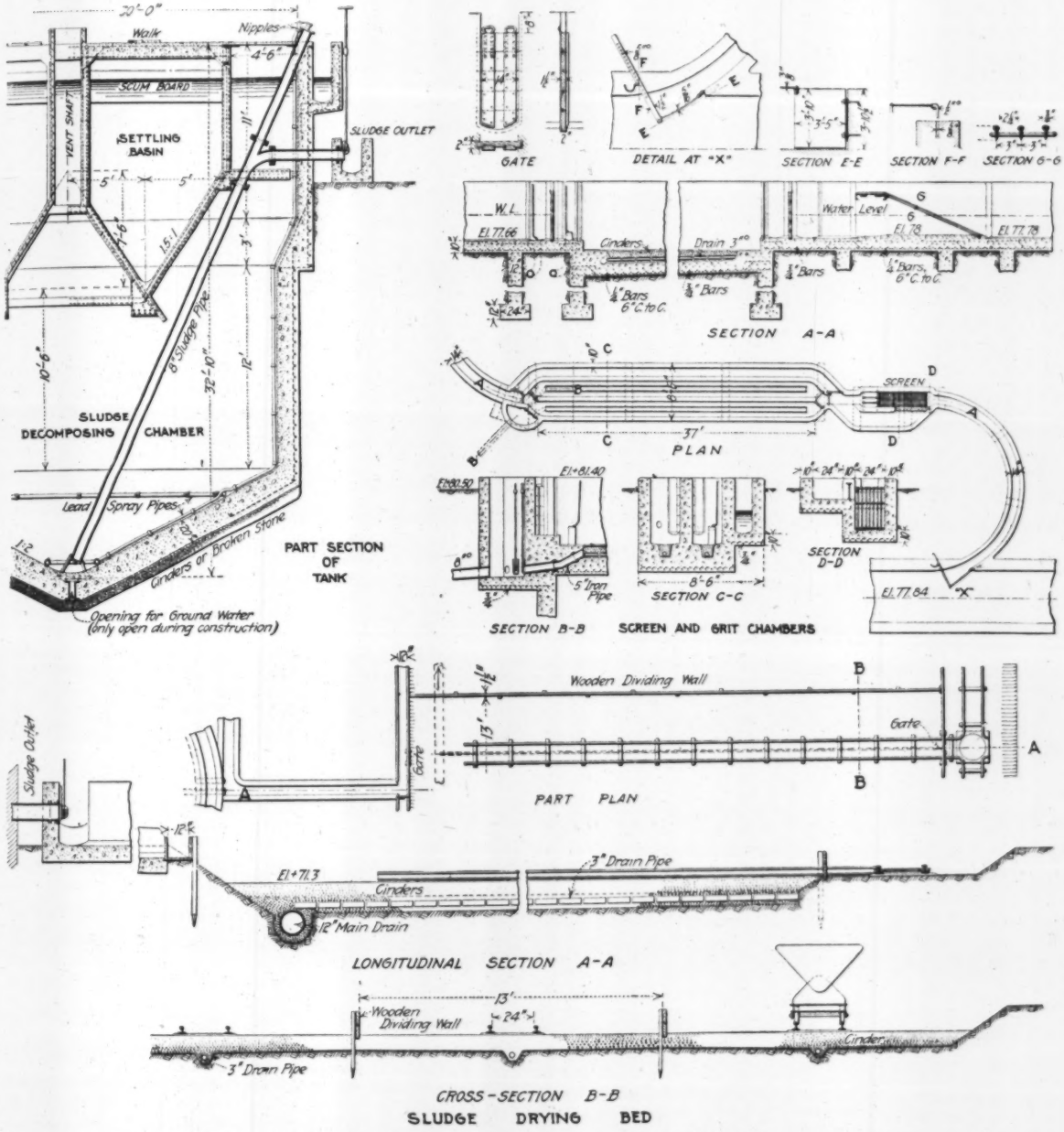


FIG. 2. DETAILS OF IMHOFF TANK AND ACCESSORIES AT MANCHESTER, ENGLAND
(Daily capacity, 750,000 Imp. or 900,000 U. S. gal.)

The plant will be operated in two ways—(1) in the manner adopted by the Emschergenossenschaft, or with a varying velocity due to increased volumes up to three times the dry-weather flow; and (2) with a uniform rate of flow through the sedimentation chamber. The second method will be followed for a time, to determine the best velocity for sewage of a certain composition throughout the day.

could buy the material from other mills. The court found in favor of the company, and this decision has been sustained on appeal.

The higher court says that the effect of the strike-and-fire clause was "not to extend the time of performance beyond the time limit, but wholly to relieve the defendant from the obligation to furnish the flooring called for by the contract . . . and the judge was warranted in finding that by reason of the destruction of the mill by fire it was impossible for the defendant to perform the contract according to its terms."

Concrete Lining for Irrigation Canals at Burbank, Wash.*

By Elbert M. Chandler†

In March, 1910, the Burbank Power & Water Co., the predecessor of the Burbank Co. constructed 3000 ft. of plain concrete lining on a steep side hill that had become honeycombed with gopher holes. This lining was made 2½ in. thick on the bottom and 3 in. on the sides, and a coping 6 in. wide on each outer edge was supplied. The predominating idea in adopting the bottom thickness employed was to give the bottom sufficient slab strength to permit of unshod teams entering the canal for purposes of cleaning. The reason a greater thickness was adopted for the sides than the bottom was because, other things being equal, the sides usually give way first on account of being less fortified against frost heaving. The object of the coping at each outer edge was to prevent the accumulation of moisture, and consequent damage to the lining, by expansion from freezing, by keeping the soil immediately against the upper parts of the side slopes dry.

No underdrainage was provided because the nature of the subgrade was such as to carry away readily any accumulation of water from whatever cause. Water was held in the canal bed for two weeks continuously prior to lining in order to secure perfect settlement of the foundation. This was regarded as a very important point since subsequent settlement from the sweating of the lining, if the subgrade were not previously settled properly, would leave the lining unsupported and precipitate its destruction.

The work of trimming the subgrade and laying the lining was done by contract on the basis of $12.50 per cu.yd. The specifications required the careful and accurate trimming of the canal bed to permanent subgrade, the laying of the concrete in strips 3 ft. wide clear across the canal, the use of no forms (but templates to be used as guides) the employment of no expansion joints by permitting the natural joint between strips of concrete to take up the expansion and contraction due to temperature changes, the subsequent painting of the concrete with a 1:1 cement-sand mortar, and the setting of the whole under wet burlap. The side slopes were 1½ to 1 and the lining was dressed into shape with square concrete shovels. The reason the flat side slopes were selected was because of the original canal slopes and the difficulty of laying up concrete on steeper sides without forms.

The lining described has now passed through five seasons and has maintained a perfect state of preservation. What small temperature cracks developed at the joints were soon filled with silt from the water and are quite watertight.

Based upon the experience and studies of the work in 1910, the Burbank Co. undertook the lining of 8250 ft. of its low-level canal in March, 1913, with its own forces. Construction methods were carefully planned in advance and followed without variation. As before, the canal bed was settled with water for two weeks. The soil was so permeated with gopher holes that this work was attended with great difficulty, the canal being divided into short compartments, and water permitted to run from the upper compartments to the lower for filling; even then several serious canal breaks occurred.

After canal settlement, line and grade stakes were set every 5 ft., each stake being a hub with center marked and its top set to the final concrete grade. The predetermined width of the strips was 5 ft. The predetermined thickness of concrete lining was 0.2 ft., the base 6 ft., depth of canal 3 ft., and side slopes 1½ to 1. The carrying capacity was 60 sec.-ft.

With the line and grade established, templates were constructed having the exact thickness of the proposed lining and the exact shape of the finished canals. With the line and grade stakes and a carpenter's level, it was possible for the workmen to trim the subgrade precisely. This work was carried on a few hundred feet in advance of the canal lining. Accurate trimming was essential to secure uniform thickness of lining.

Sand and gravel were found about one mile from the center of the work, and after being run through a 1½-in. rotary screen were in correct proportion. This was hauled by contract at $1.75 per cu.yd. measured in the finished lining and placed in piles above the canal 150 ft. apart and 15 ft. back from the slope of the canal, the slope of the ground above the canal not being very great. Experience subsequently proved that the piles of gravel might better have been 200 ft. apart.

Two steam-driven concrete batch mixers mounted on trucks and equipped with side loaders were started at the center of the canal to be lined, each working away from the other and endeavoring to be first to attain its own end of the lining. The mixers were pushed from pile to pile on plank runways by the men, the mixers being on the upper side of the canal at all times and passing between the canal and the line of piles of gravel. For each mixer outfit, a movable trough or chute was provided for taking the discharge of the mixer and depositing it in the concrete carts in the bottom of the canal. The lining was laid at two points for each mixer, starting 75 ft. on each side and working toward the mixer. Plank runways in the canal bed were provided and one concrete cart for each laying gang was employed.

One barrel of cement was used to 1 cu.yd. of finished concrete, or about 1 to 7, made as wet as the side slopes would permit. Two men in each laying gang placed the concrete roughly with square-pointed shovels. One man helped dump the concrete carts in the bottom of the 5-ft. strip being lined, and between times made ready the next strip and dampened the subgrade, while a fourth man in each laying gang troweled the rough concrete into finished shape.

Three men were required for each mixer to supply the raw materials to the machine—one man for fireman and engineer, one man to dump the mixer, and one man to hoe the concrete down the chute. An additional man covered the finished lining with wet burlap strips and kept moving them forward.

The water was brought an average of 1½ mi. by three four-horse teams hauling 400-gal. tanks on wagon trucks. The exact amount of coal required at each setting of the mixer was predetermined and left there in advance in sacks.

*From a paper read before the Washington Irrigation Institute, December, 1914. The remainder of the paper dealt with lining 3000 ft. of canal with "toncan" metal shaped to a semicircle with a diameter of 6 ft. 4½ in. (see "Engineering News," Sept. 4, 1913). The only maintenance work on the steel-lined canal, which has been used two seasons, has been the tightening of the bands made necessary by the shrinking of the carriers.

†Vice-President and Manager, Burbank Co., Burbank, Wash.

The laying of this 8250 ft. of concrete lining was completed in 14 working days. As much as 900 ft. in one day was accomplished. The men became very skillful in moving the machines and were able to lose not over 15 minutes' time at each moving.

The cost of the actual concrete was as follows:

	Per cu.yd.
Sand and gravel	$1.75
Cement delivered	2.65
Water	0.25
Coal	0.10
Labor mixing concrete	0.65
Labor laying concrete	0.88
Superintendence	0.31
Total	$6.59

In addition to the above, the cost of equipment, less its salvage value, was 32c. per cu.yd., the cost of trimming the canal bed was 72c. per cu.yd., and the engineering was 32c. per cu.yd. This made a grand total of $7.95 per cu.yd. or $1.10 per lin.ft. of canal.

The cost of cement was $2.25 per bbl. f.o.b. Burbank. Common labor was paid 25c. per hour, with a bonus of 2½c. for staying until the job was finished, and the finishers and firemen were paid 27½c. per hr., with a bonus of 2½c. under the same conditions. With but few exceptions, we were required to pay the bonus, and it was a good investment, as it overcame the great demoralizer of day-labor work of this kind—constant changing of personnel. The incentive to do fast work was created by the two mixer gangs—racing for the finish. To verify the efficiency of the above operations, the labor costs for the mixing and laying in the above costs totals only $1.33 per cu.yd., just one-half the cost of the cement.

The lining last described has passed through two irrigation seasons and one winter without the slightest indication of undue cracks or disintegration in any manner, presents a very true alignment and even cross-section, and is in every way very satisfactory. It should be noted that expansion joints were not used and that the interior concrete surface was not plastered with a mortar coat.

Octagonal Framed Dome, San Francisco Auditorium

The Exposition Auditorium is a sister building to San Francisco's new City Hall. The Auditorium occupies a westerly block in the Civic Center bounded by Hayes, Polk, Grove and Larkin St.*

The main ground plan (see Fig. 2) is 233 ft. 6 in. by 390 ft. 9 in., neglecting offsets. The four-story side wings, 104 ft. 5 in. by 233 ft. 6 in., provide exhibit and lecture halls. A front portion on Grove St., 42 ft. by 181 ft. 11 in., contains main entrances, stairways and corridors. Within the three features just named is inclosed the unique main unit of the building, the dome-covered auditorium, seating 5000 on the ground floor and 5000 in the balconies. Its great U-shaped balconies project into the three sides in the second, third and fourth stories.

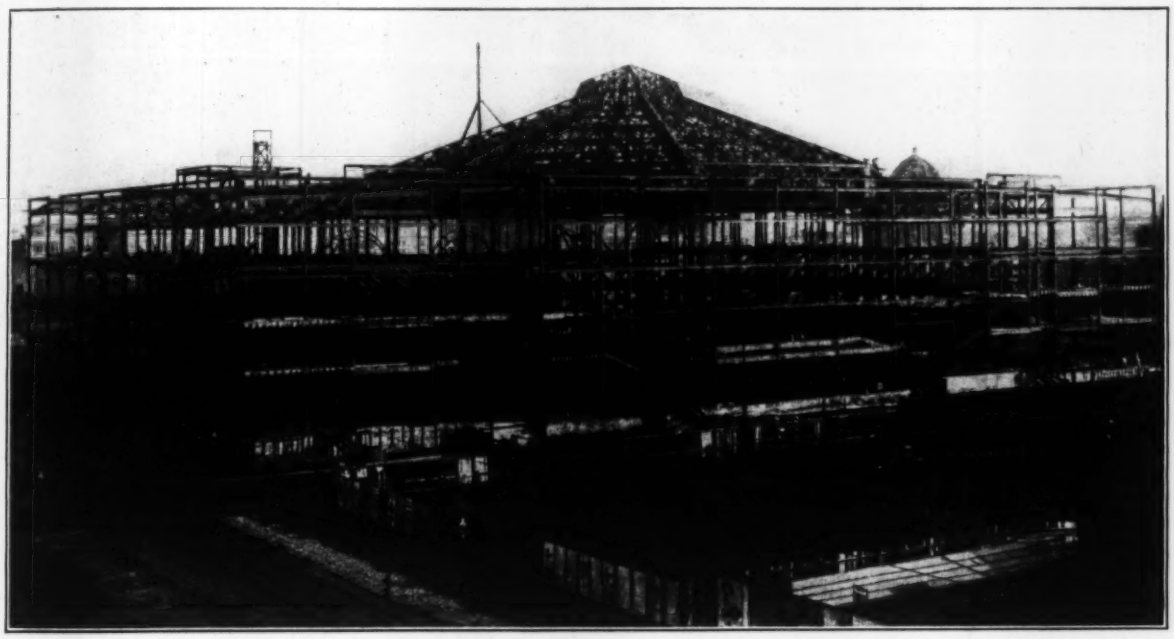


FIG. 1. SAN FRANCISCO AUDITORIUM; STEEL FRAME WITH 190-FT. OCTAGONAL DOME
(View taken Feb. 19, 1914; dome falsework removed a week before)

The roof of the hall is covered by an octagonal steel dome 189 ft. 11 in. in diameter. Its structural-steel framing is visible from below. The timber roof sheathing is covered with Spanish tile. The central portion of the dome (within the crown girder) is covered with glass skylights instead of wood and tile.

The dome frame is designed to carry the actual dead load plus a vertical wind pressure of 20 lb. per sq.ft. Because of the low rise of the dome and since it is surrounded on three sides by substantial buildings, no wind acting against a vertical area was considered. The effect of an inclined wind or of a partial wind loading was neglected.

The eight ribs of the dome (Fig. 3) are retained in place by heavy bounding trusses arranged in an octagon; eye-bars inclosed in their chords make this octagon serve

*The land for the Auditorium was given by the City and County of San Francisco. The building as originally planned was to be the sole gift from the Panama-Pacific Exposition Co., which agreed to furnish the sum of $1,000,000. The structure as finally accepted will cost more than $1,200,000. The additional money is being appropriated by the city.

as a hoop. In the calculation it was assumed that the thrust stresses in the ribs might be carried equally by the two chords of the octagon trusses.

In determining the main radial and circumferential dome stresses a method of analysis was used similar to that described by E. Schmitt, Trans. Am. Soc. C. E., Vol. 52, June, 1904, p. 62, in an article entitled, "The Theory of the Spherical Dome with a Homogeneous Surface and the Framed Dome; also Notes on the Construction of Metal and Masonry Domes." Calculation by this method showed a total circumferential tension of 728,000 lb. in the two eye-bar rings. The thrust in the ribs ranges from 62,000 lb. at the lantern to 597,000 at the outside. The interior circumferential girders take compression only, ranging from 75,000 lb. in the crown girder to 196,000 in the girder next the octagon truss. The intermediate radial frames take no computed stresses; they are bracing.

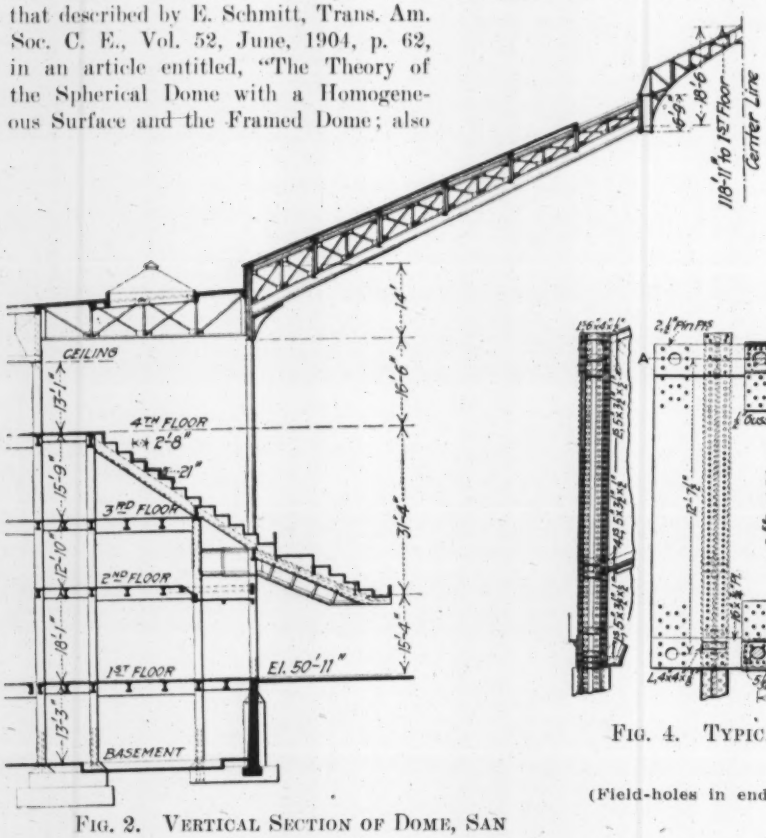


FIG. 2. VERTICAL SECTION OF DOME, SAN FRANCISCO MUNICIPAL AUDITORIUM

The radial and octagonal trusses are constructed mainly of angle members, except that the outer panel of the radials has a plate web. The lantern, or crown girder, is made purposely heavy. It supports a light lantern frame, 39 ft. 11 in. in diameter.

Perhaps the most interesting feature of the dome is the design of the encircling octagon trusses (Fig. 4), with their contained eye-bar hooping. These trusses, 78 ft. 8 in. long between main supporting columns, 14 ft. deep over all, consist of heavy box-girder chords inclosing eye-bar chains. The web members are angle struts in a double system. These trusses support large vertical dead and live loads coming from adjoining roof construction; the trusses exclusive of eye-bars are designed to take these vertical loads.

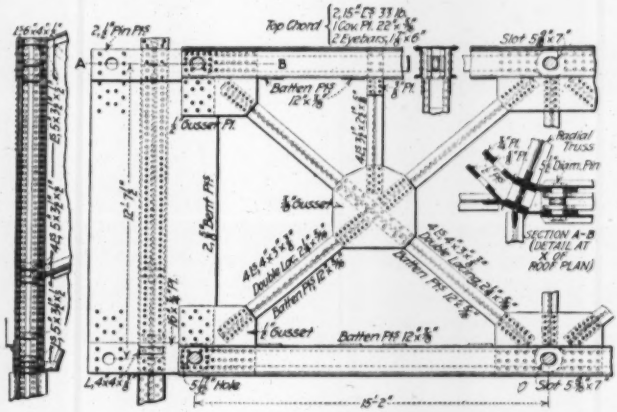


FIG. 4. TYPICAL DETAIL OF OCTAGON TRUSS AND EYE-BARS

(Field-holes in end gussets drilled after dead load of dome was placed)

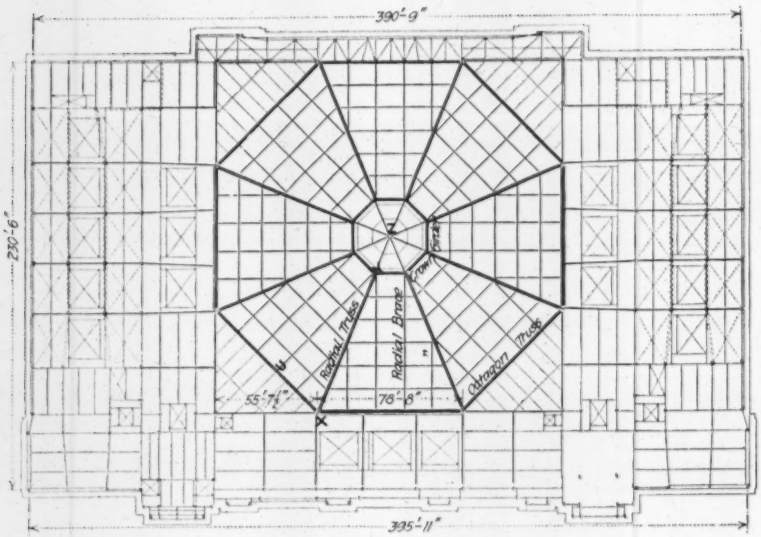


FIG. 3. PLAN OF DOME

The distances between centers of pinholes in the eye-bar chains were specified to be $\frac{3}{32}$ in. shorter than the length of the corresponding truss-panel. The column gusset-plates were specified to be drilled in the field after the full dead load on the dome roof brought its stresses upon the eye-bar chains.

Fig. 1 is a view of the steel frame of the building during erection. A week earlier the falsework tower supporting the crown girder had been removed. When this view was taken the rivet-holes in the column gussets at the octagon trusses were being drilled and these connections completed.

The field erection proceeded as follows: The octagon trusses were first lifted into place. The crown girder was then placed upon the central falsework tower at a building was the Lindgren Co., of San Francisco; the C. A. Blume Construction Co. had the contract for steel erection.

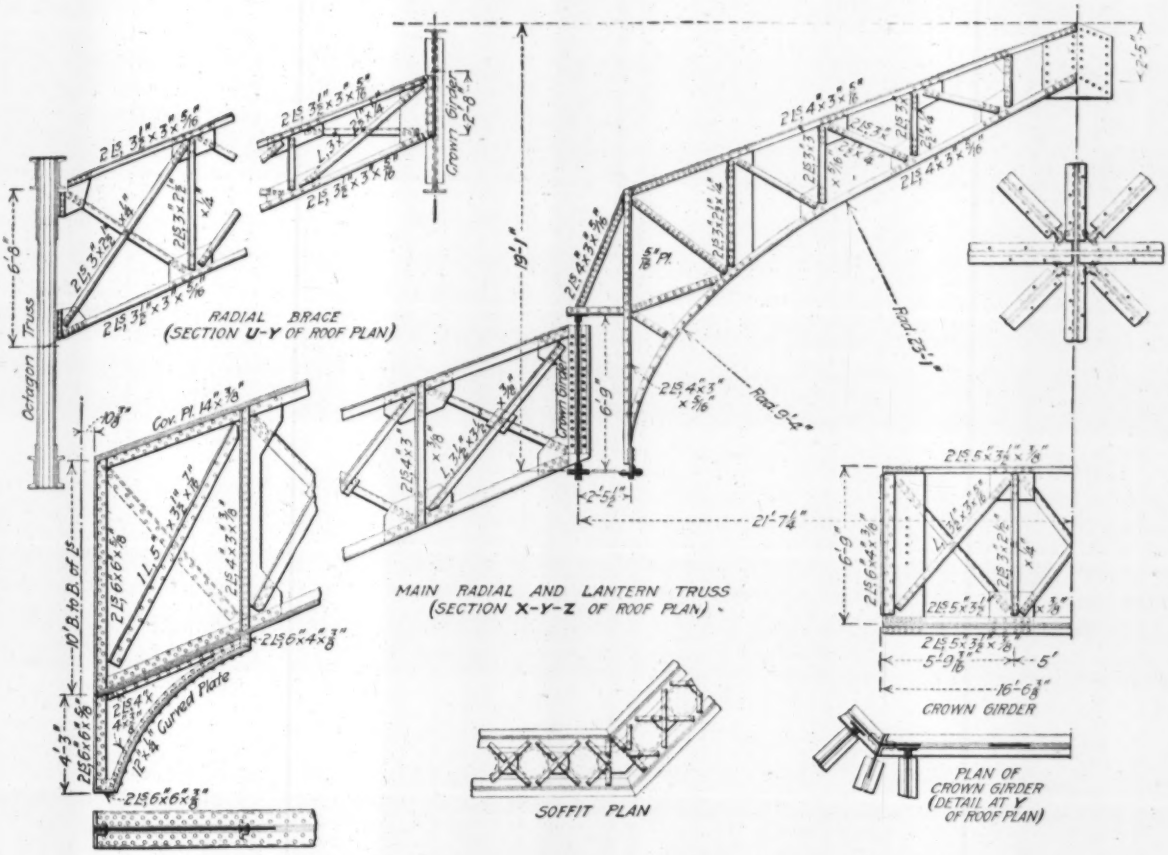


FIG. 5. DOME-FRAMING DETAILS: RADIAL GIRDER, RADIAL BRACE, CROWN RING

level 4 in. higher than its final position. The main radial trusses were next hoisted into place. At any stage of the erection a lateral balance was maintained with respect to the central tower. The circumferential girders were placed, starting with the one next the octagon truss and working toward the crown; the bracing ribs were placed in the same order. As the dome took more and more of its dead load, the crown was gradually lowered by releasing blocks; as already mentioned, the rivet connections on the main columns were the last work completed. The dome settled ¼ in. at the crown below its calculated position.

Originally, it was intended to cover the dome with a reinforced-concrete slab and copper outside surface, but to reduce cost the change was finally made to 3-in. t. & g. Oregon pine carrying tile. This lighter roof covering greatly decreased the stresses and therefore the structural steel tonnage in the dome. For the first design the dome contained 610 tons of structural steel with a maximum hoop tension in the octagon truss of 1,095,000 lb. The revised dome (as here described) contains 443 tons of structural steel. One of the octagon trusses, including pins and eye-bars, weighs 24 tons.

The total steel tonnage in the building is 3305; the cast-iron bases adds 105 tons. The steel frame was manufactured at Gary, Ind. The general contractor for the

Prof. Charles Derleth was consulting engineer to the board of architects which designed the building. H. J. Ludwig was chief designer, under his direction.

⁂

The Use of Large Jaw Crushers, capable of taking in very large blocks of rock, is now accepted as standard practice in all large-scale rock- and ore-crushing operations. While these large and very heavy machines are expensive in first cost, they enable the rock to be dumped into the crusher just as it comes from the quarry, including pieces as large as an ordinary steam-shovel dipper can handle, and thus save the expense and delay of breaking up the large blocks by drilling or dobe-shooting. At the newly completed plant of the New Haven Trap Rock Co., at North Branford, Conn., which was described in "Engineering News" of Mar. 18, the rock from the quarry is dumped into a Buchanan jaw crusher with an opening 48x72 in. The Bethlehem Chili Iron Mines has just had completed by the Power & Mining Machinery Co. a jaw crusher with an opening 60x84 in., which is designed to handle the very hard Chilian iron ore, some samples of which show a compressive strength as high as 44,000 lb. per sq.in. This machine is one of the heaviest rock crushers ever built, its total weight being 525,500 lb. The same manufacturer has recently shipped to a customer on the Pacific Coast a jaw crusher with a 66x84-in. opening, and it has another of the same size in its shops. A jaw crusher with even a larger opening is now being built by the Traylor Engineering & Manufacturing Co. for the Rockland Lake Trap Rock Co., the jaw openings measuring 66x86 in. While these huge crushers are high in first cost, they have an enormous output and a long life. The Power & Mining Machinery Co. informs us that a 60x84-in. machine, built by it for the Birdsboro Stone Co., of Birdsboro, Penn., five years ago, has crushed about 2,000,000 tons of trap rock, and shows very little wear.

High Concrete Retaining Wall with Structural Reinforcing

A reinforced-concrete retaining wall of the buttress type has recently been completed on the approach to Lock 1 of the new Welland Ship Canal, at Port Weller, the Ontario end of the canal. In addition to being very high, 41½ ft. from footing to coping, the wall is remarkable in the use of a structural steel frame for the counterforts, which frame serves not only to reinforce the counterforts but also holds the rod reinforcement and the concrete forms.

The details of the wall are shown in Fig. 2. It consists of a solid reinforced-concrete face, base and coping (which extends inward of the face to form a walkway), with counterforts every 12 ft. The structural frames, in each counterfort, are of angle-irons punched to carry the horizontal reinforcement, which is continuous from face wall into

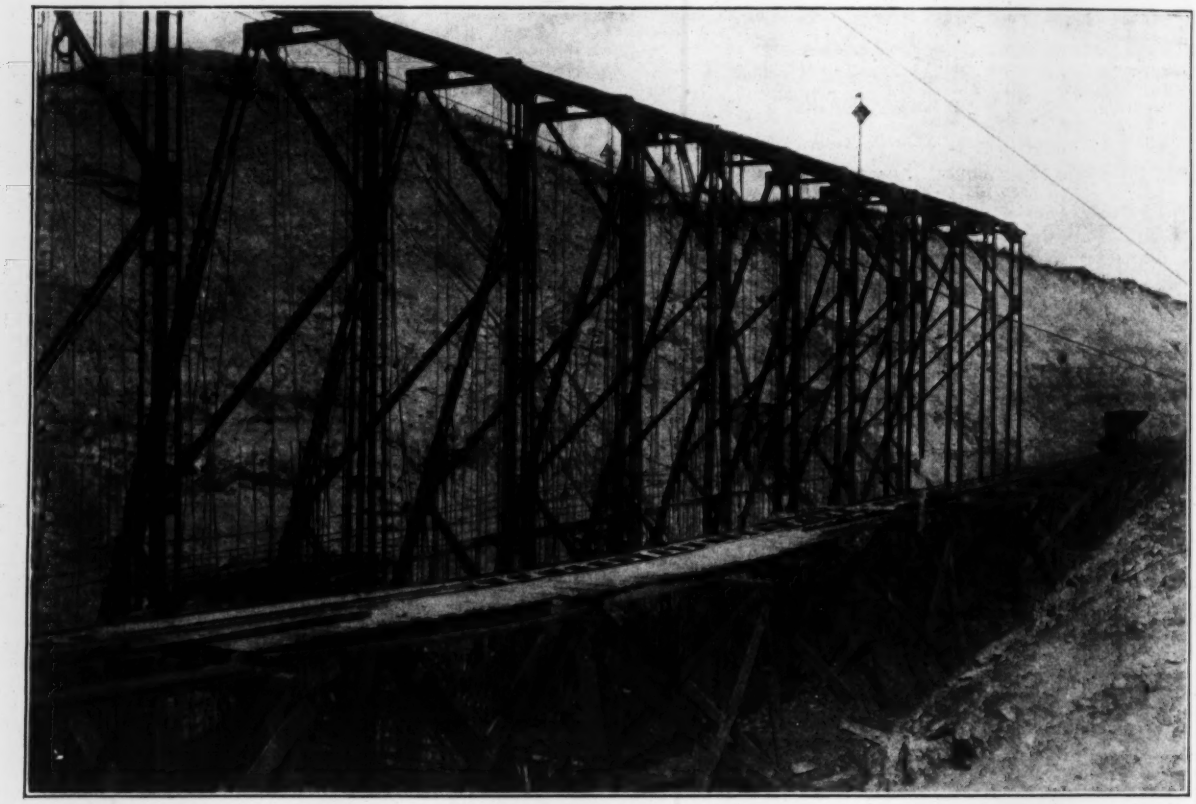


FIG. 1. FRAMEWORK REINFORCEMENT OF RETAINING WALL, APPROACH TO LOCK 1, WELLAND SHIP CANAL

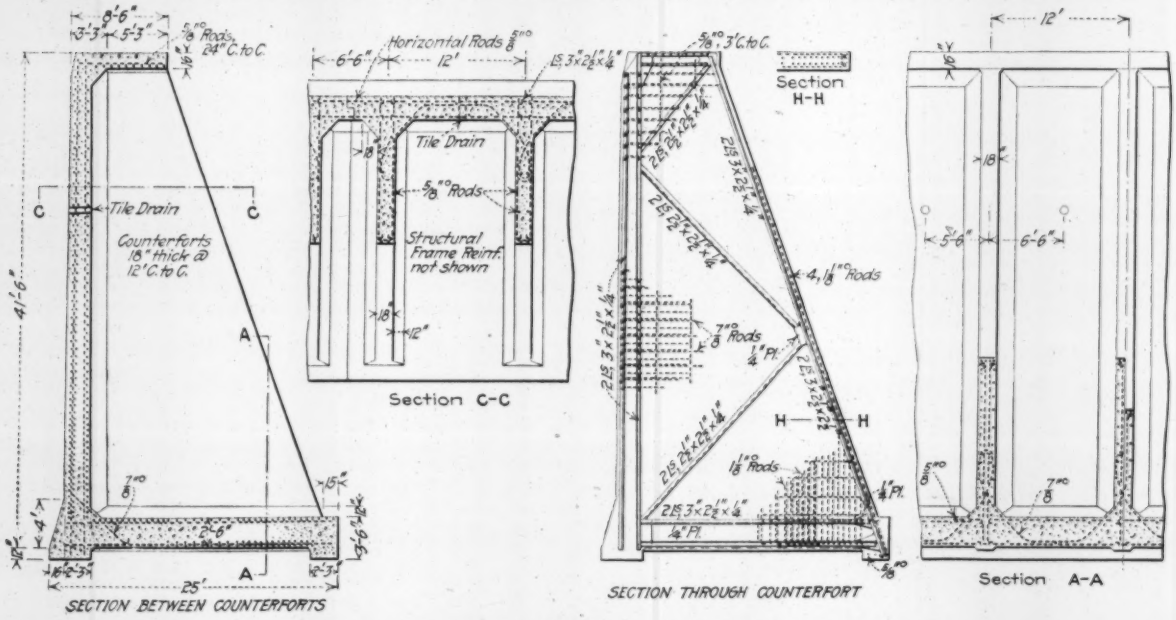


FIG. 2. DETAILS OF WELLAND CANAL RETAINING WALL

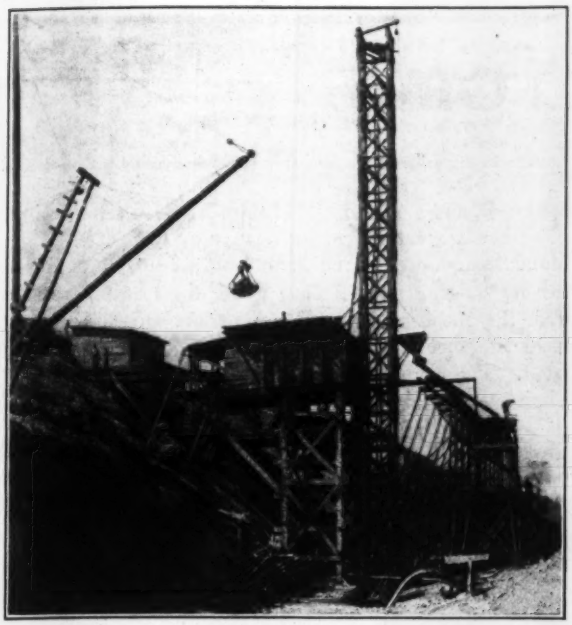


FIG. 3. CONCRETE MIXING PLANT FOR RETAINING WALL

counterforts. Fig. 1 shows the frames in place with some of the rod reinforcement threaded through them. The forms are afterward hung from the framework.

The frames were made quite heavy because they were designed to take the full 41 ft. lift of concrete in one pouring. In construction, however, the wall was cast in several lifts, so that so much metal was not required. Except for this the method proved eminently successful, particularly in insuring stiffness of the wall during construction without much temporary bracing.

For concreting the wall a very efficient plant was installed. A Hains gravity mixer was located about the center of the length of the wall, where it was easily loaded by derrick from the adjacent high-level railway (Fig. 3). Concrete from the bottom, or delivery end, of this mixer was run into an elevator, whence it was lifted to be dumped into a hopper and chute leading to another hopper with a bottom dump located on a frame just outside of the wall forms. All of the preceding equipment was stationary, but alongside of the wall was a trestle on which ran a car which took concrete from the last-noted hopper and dumped it through another chute to its proper place in the forms. The number of chutings given each batch of concrete should be especially noted. The view in Fig. 1 shows the trestle paralleling the forms and the concreting car.

The Welland Ship Canal is being built under the direction of J. L. Weller, Engineer in Charge. The contract on which is located the wall noted here is held by Lane Bros.

Steel Wagon-Shed

A steel wagon-shed has been built recently at Wheaton, Ill., for the protection of the city's wagons, sprinkling carts and other equipment. It is 50x21 ft. in plan, with a clear height of 10 ft. The posts are connected along the sides and rear by three horizontal girts, and the ends have also diagonal bracing. The front (being left open) has only the top girt. The sheathing of ends, rear and roof is galvanized iron of No. 24 gage. The

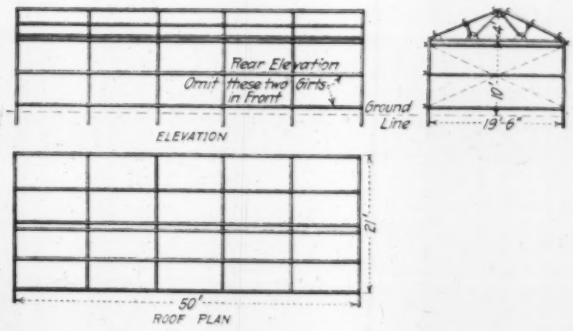


FIG. 1. GENERAL DESIGN OF STEEL SHED FOR SHELTERING CITY WAGONS

construction is shown clearly in the drawings, Figs. 1 and 2.

The cost was about as follows: Steel, $214; concrete footings and steel erection, $64; lumber and carpenter work, $24; galvanized sheathing, $108; placing sheathing

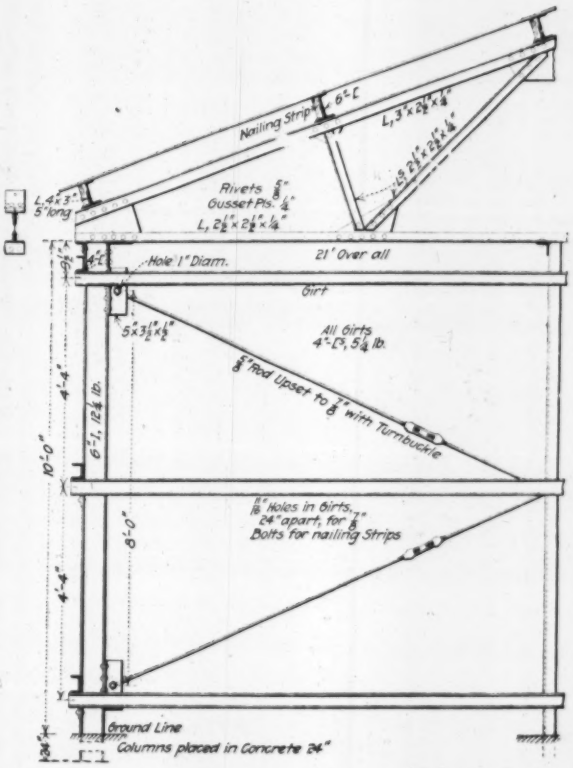


FIG. 2. DETAILS OF STEEL WAGON-SHED

and gutters, $45; total, $455. The structure was designed by Laverne J. Ruddock, City Engineer, and was built by day labor under his direction.

City Pavements in the Northwest—A Western correspondent informs us that at the present time creosoted wood block is the most popular paving material in the Northwestern cities, and very large areas are being thus paved in St. Paul, Minneapolis and nearby cities. In Northwestern Canada during the last five or six years a large amount of asphaltic-concrete pavement has been laid in preference to sheet asphalt, on the theory that this type of pavement stands extremes of cold weather more satisfactorily than sheet asphalt.

Field and Office

Unmanageable Coffer-Dam Leak Handled by Freezing

A leak in the steel sheetpile coffer-dam of the O'Rourke Engineering Construction Co. for the west river pier of the Detroit-Superior arch bridge over the Cuyahoga River at Cleveland, Ohio, caused so much trouble and expense in attempts to stop it that finally the freezing process was resorted to. This process proved fully successful. Closing the leak was necessary because the county engineer required the contractor to place the pier concrete in the dry.

The first coffer-dam inclosure of the site was made with 70-ft. steel sheeting. At one point this piling was badly deformed, and a second line of sheeting was driven behind it. When excavation had got down near the bottom of the foundation, however, the great head of 46 ft. produced further bulging at the place where the original trouble with the sheeting occurred. A puddle-backed bulkhead inside did not stop the deformation and the clay movement. The continuing pressure could not be controlled. Then the freezing process was resorted to.

FIG. 1. DETROIT-SUPERIOR BRIDGE PIER COFFER-DAM
(Point of leak marked by arrow)

FIG. 2. INSIDE OF COFFER-DAM AFTER CONCRETE WAS PLACED
(One freezing pipe still in place, as marked by black arrow; edge of concrete marked by white arrow)

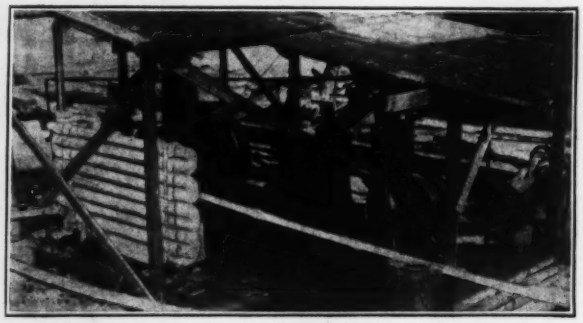


FIG. 3. THE REFRIGERATING PLANT

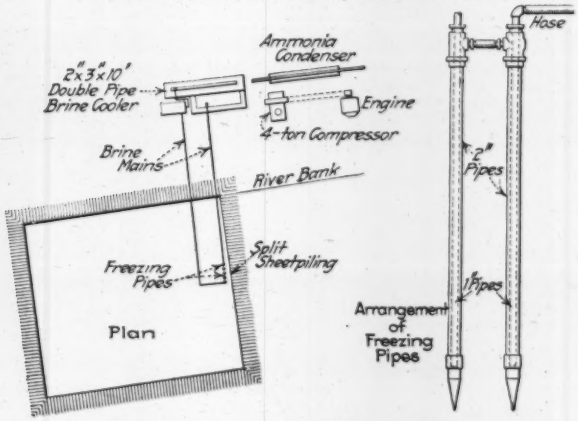


FIG. 4. SKETCH OF FREEZING-PIPE ARRANGEMENT

A refrigerator plant for eight freezing pipes was set up on the bank a few hundred feet away. It included a 10-hp. engine, an ammonia pump, a brine circulating pump and a brine cooler. The plant circulated calcium

chloride brine at 10° F., at the rate of about 30 gal. per min. The freezing pipes were constructed as shown in the sketch Fig. 3. Five days' operation of the plant froze a block of material 2 ft. thick at right angles to the wall, 4 ft. long and about 16 ft. high. Excavation was then continued and concrete deposited, keeping the refrigerating plant in operation another week. The plant was running altogether about two weeks.

The total cost of the freezing was about $1600, of which roughly one-third was a lump-sum payment for rental, installation, operation and removal of the ice machine (except for labor). Something like $10,000 was spent in the other attempts to stop the leak before applying the freezing process.

R. H. Whipple, engineer for the Phoenix Ice Machine Co., Cleveland, worked out the details of the plant. John F. O'Rourke, of the contracting company, is responsible for the use of the method.

Azimuth Observations on Polaris by Daylight

By Robert V. R. Reynolds*

Polaris may always be found in clear weather as soon as the sun has set, and very frequently for five or ten minutes before sunset or after sunrise. It is stated on good authority that under very favorable conditions an observation has been successful as late as 10 a.m. In the northern parts of the United States the cross-wires may remain visible for a long time after sunset.

For the novice it is often difficult at the first few attempts to see Polaris while the sky is still bright, but having once found the star, which appears as a small white dot in the field, he will never thereafter feel in doubt. Tabulated settings (such as accompany this article) sufficiently accurate to bring the star into the field are required, and there remain several factors which must be given consideration before success can be assured:

TABLE TO FIND POLARIS BY DAYLIGHT

Hour Angle of Polaris Approximate† (Use Suitable Interpolations)	Azimuth Setting N.E. or N.W. Depending upon Position of Polaris E. or W. of Meridian	Altitude Setting Latitude Plus or Minus the Tabulated Quantities	
0.0 or 12.0	00′		1° 08′
0.5 or 11.5	12′		1° 07′
1.0 or 11.0	24′		1° 06′
1.5 or 10.5	36′		1° 04′
2.0 or 10.0	47′		1° 00′
2.5 or 9.5	57′		55′
3.0 or 9.0	1° 06′	Add to latitude for hour angles less than 6.	50′
3.5 or 8.5	1° 15′		43′
4.0 or 8.0	1° 22′		35′
4.5 or 7.5	1° 27′		27′
5.0 or 7.0	1° 30′		18′
5.5 or 6.5	1° 33′		09′
6.0 hours	1° 35′	Subtract from latitude for hour angles exceeding 6.	00′

†The hour angle used as the argument in this table needs only to be approximate. If it is correct within 5 min. of time, sufficiently accurate settings will be indicated provided interpolation is made. Hence, there is no need of correcting for longitude until the surveyor has made the observation and is preparing to enter the table of Azimuths of Polaris.

The table is computed for a mean latitude of 42°, but purposely modified slightly to make it more useful along the 49th parallel, where much of the Forest Service work is being done. It is accurate enough to bring Polaris within the field of an ordinary transit between latitudes of 10° and 58° N. It is not for use to determine the true azimuth after the observations.

1. A slight haziness, which may hardly be obvious to the eye, is sufficient to conceal the star until darkness comes on.

2. The telescope must be in exact focus for celestial objects. This may be accomplished either by focusing at night upon the moon and making a slight scratch upon the objective slide to show the point to which it should be extended, or the surveyor may focus at the time of observation upon a well defined object 3 or 4 mi. distant, which focus will usually be found sufficiently close. Accurate focusing is one of the most important factors in finding the star.

3. For the purpose of cutting off objectionable light, the sunshade should always be attached. Certainty of finding the star is assured by throwing a coat or other dark cloth over the head when searching through the telescope, as a photographer uses a focusing cloth.

4. An approximate meridian must be had, from which the azimuth settings are turned off. Commonly, the surveyor will already have such a meridian from his backsight. Otherwise, a meridian determination from a solar attachment in reasonable adjustment will suffice. Sometimes, when the magnetic declination is closely known, it will even be possible to turn upon the star from the needle. A reference meridian which is true within 5′ or 10′ will be precise enough to locate the star when the table of approximate settings is used.

Polaris having been found, the angle from the reference mark to the star should be measured twice, the second time with the telescope inverted. The mean time of observation and the mean angle are then used to find the azimuth of the mark by the simplified hour-angle method. There is practically no chance that any other star will be seen and mistaken for Polaris.

*Forest Examiner, U. S. Forest Service, Washington, D. C.

Arrangement of Scow Bridge for Building Falcon River Dike, Winnipeg District

Referring to the description of Cook Construction Co. & Wheaton Bros.' method of building a rock-fill breakwater at Halifax by using a dumping bridge whose outer end is supported on a scow, M. V. Sauer mentions that the same method was used in the Winnipeg Water District to build a diversion dike on Falcon River.* The distinctive feature of the Halifax case, however, was the use of the dumping-bridge-and-scow method in *tidal water*. This required that the scow be arranged with ballast compartments and pumps, so that it could be kept at the same height with respect to the breakwater as the tide rose and fell. The Halifax contractors worked in a 6-ft. tide with no trouble by this method. At Winnipeg there was no tide to complicate the problem.

The details of the Winnipeg arrangement are interesting. When bids were called for, the engineers of the water district suggested that the scow method be used; nevertheless, all bidders but one figured on dumping from a trestle. Tomlinson & Fleming, the successful bidders, saved over $25,000 to the district by using the scow method.

The contractors used two scows in tandem. The track on the boats consisted of 90-lb. rails on a framework whose sills rested on, but were not fastened to, the deck. To move forward, the scows were outhauled under the rail-supporting frames by a cable passing forward over a snatchblock at the outer end of the rail and back to the dinkey. Then the inshore ends of the 90-lb. rails were moved to the outer end of the scows in the new position, and the gap filled in with the regular 60-lb. rail as used in the rest of the supply track on the finished fill.

The dike, 8000 ft. long, containing about 230,000 cu.yd. of gravel, progressed rapidly, being practically completed in four months. The greatest depth of water was about 25 ft. Material was obtained from a gravel pit adjacent to the north end of the dike.

*See view in issue of Feb. 4, 1915, p. 230, showing the dumping bridge and a train of dump cars being unloaded.

Chart of Equivalent Uniform Loads for Railway Bridges

By D. B. Steinman*

The accompanying chart has been devised for the purpose of minimizing the time and effort required for figuring stresses in girders and trusses. It is constructed for Cooper's loading and gives directly the equivalent uniform load for maximum moment or shear at any point of any span up to 1000 ft. It is necessary merely to multiply this equivalent uniform load by a simple dimensional factor to obtain the maximum value of any stress. The chart can be used for bending moments, shears, chord stresses, web stresses, counter stresses, end reactions, floor-beam reactions, etc. With its use all of the stresses in an ordinary truss can be found in a few minutes.

The chart is based on the principle that the equivalent uniform load for any stress depends only on the horizontal lengths, l_1 and l_2, of the two segments of the influence triangle for that stress. Accordingly, the diagram consists of curves of constant equivalent uniform load plotted with l_1 and l_2 as coördinates. For $l_1 < 142$ ft., the curves were plotted by using the writer's formula for equivalent uniform loads,†

$$q = 6000 + \frac{23{,}040{,}000}{l_1\,(l_1 + l_2)}$$

For smaller values of l_1 the values of q were derived from previously published moment-tables and from values of moments specially computed for this purpose. The sketches on the chart, with subjoined formulas, indicate how the values of q are used in finding the various kinds of stresses.

Bending Moments and Chord Stresses

For bending moments and chord stresses, l_1 and l_2 are simply the distances of the center of moments from the ends of the span (see also Fig. 1). The corresponding value of q is read directly from the chart and multiplied by $\frac{1}{2}\, l_1\, l_2$ to give the maximum bending moment.

Ex. 1. To find the maximum bending moments in a plate-girder span 88 ft. long:

At the center, $l_1 = 44$, $l_2 = 44$; $\therefore q = 7950$; $M = \frac{1}{2}\, q\, l_1\, l_2 =$ 7,692,000 ft.-lb.

At the ¼-points, $l_1 = 22$, $l_2 = 66$; $\therefore q = 8200$; $M = \frac{1}{2}\, q\, l_1\, l_2 =$ 5,950,000 ft.-lb.

Ex. 2. To find the maximum chord stresses in a span of nine 36-ft. panels:

At L_1, $l_1 = 36$, $l_2 = 288$; $\therefore q = 6870$; $M = 35{,}630{,}000$.
At L_2, $l_1 = 72$, $l_2 = 252$; $\therefore q = 6750$; $M = 61{,}100{,}000$.
At L_3, $l_1 = 108$, $l_2 = 216$; $\therefore q = 6650$; $M = 77{,}600{,}000$.
At L_4, $l_1 = 144$, $l_2 = 180$; $\therefore q = 6490$; $M = 84{,}100{,}000$.

Dividing these values of M by the respective lever-arms, we obtain the maximum stresses in all the chord members and the end posts of the truss.

Shears and Web Stresses

For the maximum shear in any panel of a truss, l_2 is the distance to the loaded end of the span and l_1 is a fraction of the panel as indicated in the sketch Fig. 2. The value of l_1 is given exactly by $l_2 \,/\, (n - 1)$, where n is the number of panels in the span. The corresponding value of q is read from the chart and multiplied by $\frac{l_1 l_2}{2p}$ to give the maximum shear in the panel.

Ex. 3. To find the maximum shears and web stresses in a double-track Pratt truss having seven 28-ft. panels:

Panel 0-1, $l_2 = 6p = 168$, $l_1 = l_2 \div 6 = 28$, $\therefore q = 7440$; $S = \frac{1}{2}\, q\, l_1 .6 = 625{,}000$.

Panel 1-2, $l_2 = 5p = 140$, $l_1 = l_2 \div 6 = 23.3$, $\therefore q = 7700$; $S = \frac{1}{2}\, q\, l_1 .5 = 449{,}000$.

Panel 2-3, $l_2 = 4p = 112$, $l_1 = l_2 \div 6 = 18.7$, $\therefore q = 8030$; $S = \frac{1}{2}\, q\, l_1 .4 = 299{,}500$.

Panel 3-4, $l_2 = 3p = 84$, $l_1 = l_2 \div 6 = 14$, $\therefore q = 8350$; $S = \frac{1}{2}\, q\, l_1 .3 = 175{,}500$.

Multiplying these values of S by sec a, we obtain the maximum stresses in all the diagonals and the end posts. The last two values of S also constitute the maximum stresses in the vertical posts.

Web Stresses with Inclined Top Chord

Where the top-chord is inclined, we find the ratio r between the heights of the two posts in the panel and substitute this value in the formulas given under Fig. 3. When $r = 1$, these formulas reduce to the simpler ones of the preceding case.

Ex. 4. To find the maximum stress in the diagonal indicated in Fig. 3:

Given p = 32 ft., n = 7, h = 36, h′ = 40. In this case a = 2, b = 4, r = 1.11, $l_2 = 4p = 128$; hence $l_1 = 16.05$. The chart then gives q = 7950. The maximum stress in the diagonal will then be $\frac{1}{2}\, q\, l_1\, l_2 \dfrac{\sec a}{rp} = +307{,}800$ lb.

Ex. 5. To find the minimum stress in the diagonal of Ex. 4:

For this stress, $l'_2 = 2p = 64$, $l'_1 = p - l_1 = 15.95$. For these values, q = 8400. The minimum stress will then be $\frac{1}{2}\, q\, l'_1\, l'_2 \dfrac{\sec a}{p} = -179{,}100$ lb.

Floor-Beam Reactions and Pier Reactions

The maximum reaction at the junction of any spans (Fig. 4) is given by the formula $R = \frac{1}{2}\, q\, (l_1 + l_2)$, where l_1 and l_2 are the two adjoining spans.

Ex. 6. To find the maximum floor-beam reaction between two panels 28 ft. long:

Here $l_1 = l_2 = 28$; $\therefore$ q = 8815; R = 8815 × 28 = 246,820 lb. per track. This also represents the maximum tension in the hanger $U_1 L_1$ of Ex. 3.

Ex. 7. To find the maximum reaction on a pier between two simple spans of 320 and 250 ft.:

For $l_1 = 250$ and $l_2 = 320$, the chart gives q = 6160. Hence $R = \frac{1}{2}\, .6160\, (250 + 320) = 1{,}755{,}600$ lb.

End Reactions and Shears

For end reactions or intermediate shears (Fig. 5), the influence diagram is a triangle with $l_1 = 0$. Instead of extendng the chart to include the corresponding values of q, it was found more convenient to give the latter in a separate table. To obtain the values of the equivalent uniform load for end reactions, the writer deduced the formula

$$q = 6000 + \frac{432{,}000}{l} - \frac{13{,}200{,}000}{l^2}$$

which is exact for spans over 101 ft. For smaller spans, the tabulated values of q were obtained from previously published tables. These values of q and the formula for intermediate shears presuppose wheel 2 to be placed at the section considered.

Ex. 8. To find the maximum end reaction of a 450-ft. truss:

For $l_2 = 450$, the table gives q = 6896, so that R = ½ 6896 × 450 = 1,551,000 lb.

Ex. 9. To find the maximum shears in an 80-ft. plate-girder:

At the ends, $l_2 = 80$, $\therefore q = 9315$; $R = \frac{1}{2}\, q\, l_2 = 372{,}600$.
At the ¼ point, $l_2 = 60$, $\therefore q = 9800$; $S = \frac{1}{2}\, q\, l_2\, \frac{3}{4} = 220{,}500$.
At the ½ point, $l_2 = 40$, $\therefore q = 11{,}310$; $S = \frac{1}{2}\, q\, l_2\, \frac{1}{2} = 113{,}100$.
To correct for the effect of wheel 1, we subtract 30,000 $\left(1 - \dfrac{l_2 + 8}{80}\right)$ from the last two values, or 4500 and 12,000 lb. respectively.

*Formerly Professor of Civil Engineering at the University of Idaho; Special Assistant to Gustav Lindenthal, Consulting Engineer, 68 William St., New York.

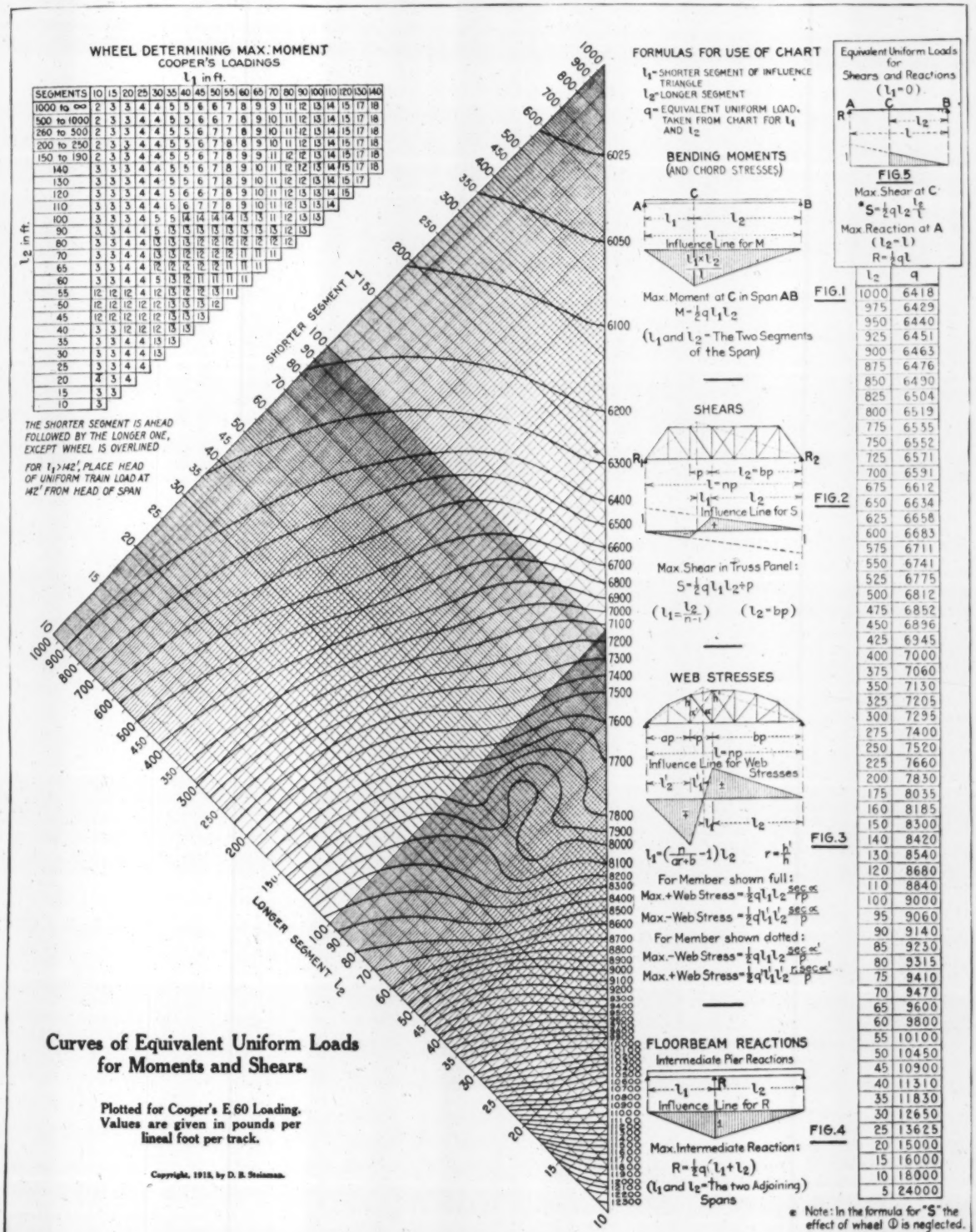
†"Equivalent Uniform Loads for Long-Span Bridges," "Engineering News," Feb. 25, 1915.

Table of Wheel Loads

In the upper left-hand corner of the chart is a table indicating the wheel to be placed between l_1 and l_2 in order to produce any of the above-cited maximum stresses. This table is inserted because many impact formulas require the length of load for which each stress is determined.

The chart is constructed for E-60 loading, as this has become the most common specification. For lighter loadings, the results must be reduced in the corresponding ratio.

All values are given per foot of track. *For single-track bridges the results must be divided by two to give the stresses in each truss.*


WHEEL DETERMINING MAX. MOMENT
COOPER'S LOADINGS
l_1 in ft.
SEGMENTS | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 | 65 | 70 | 80 | 90 | 100 | 110 | 120 | 130 | 140
1000 to ∞ | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 9 | 11 | 12 | 13 | 14 | 15 | 17 | 18
500 to 1000 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18
260 to 500 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18
200 to 250 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18
150 to 190 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 9 | 11 | 12 | 12 | 13 | 14 | 15 | 17 | 18
140 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 13 | 14 | 15 | 17 | 18
130 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 13 | 14 | 15 | 17
120 | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 13 | 14 | 15
110 | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 13 | 14
100 | 3 | 3 | 3 | 4 | 5 | 5 | 14 | 14 | 14 | 14 | 13 | 13 | 11 | 12 | 13 | 13
90 | 3 | 3 | 4 | 4 | 5 | 13 | 13 | 13 | 13 | 13 | 13 | 13 | 13 | 12 | 13
80 | 3 | 3 | 4 | 4 | 13 | 13 | 13 | 12 | 12 | 12 | 12 | 12 | 12 | 12
70 | 3 | 3 | 4 | 4 | 13 | 13 | 12 | 12 | 12 | 12 | 11 | 11 | 11
65 | 3 | 3 | 4 | 4 | 12 | 12 | 12 | 12 | 12 | 11 | 11 | 11
60 | 3 | 3 | 4 | 4 | 5 | 13 | 12 | 11 | 11 | 11 | 11
55 | 12 | 12 | 12 | 4 | 12 | 13 | 12 | 12 | 13 | 11
50 | 12 | 12 | 12 | 12 | 12 | 13 | 13 | 13 | 12
45 | 12 | 12 | 12 | 12 | 12 | 13 | 13 | 13
40 | 3 | 3 | 12 | 12 | 12 | 13 | 13
35 | 3 | 3 | 4 | 4 | 13 | 13
30 | 3 | 3 | 4 | 4 | 13
25 | 3 | 3 | 4 | 4
20 | 4 | 3 | 4
15 | 3 | 3
10 | 3
l_2 in ft.
THE SHORTER SEGMENT IS AHEAD FOLLOWED BY THE LONGER ONE, EXCEPT WHEEL IS OVERLINED
FOR l_1 > 142', PLACE HEAD OF UNIFORM TRAIN LOAD AT 142' FROM HEAD OF SPAN
SHORTER SEGMENT l_1
LONGER SEGMENT l_2
FORMULAS FOR USE OF CHART
l_1 = SHORTER SEGMENT OF INFLUENCE TRIANGLE
l_2 = LONGER SEGMENT
q = EQUIVALENT UNIFORM LOAD, TAKEN FROM CHART FOR l_1 AND l_2
BENDING MOMENTS (AND CHORD STRESSES)
Influence Line for M
Max. Moment at C in Span AB
$M = \frac{1}{2} q l_1 l_2$
(l_1 and l_2 = The Two Segments of the Span)
FIG. 1
SHEARS
Influence Line for S
Max. Shear in Truss Panel:
$S = \frac{1}{2} q l_1 l_2 \div p$
$(l_1 = \frac{l_2}{n-1})$ $(l_2 = bp)$
FIG. 2
WEB STRESSES
Influence Line for Web Stresses
$l_1 = (\frac{n}{a+b} - 1) l_2$ $r = \frac{b'}{h}$
For Member shown full:
Max. + Web Stress = $\frac{1}{2} q l_1 l_2 \frac{\sec \alpha}{p}$
Max. − Web Stress = $\frac{1}{2} q' l_1' l_2' \frac{\sec \alpha}{p}$
For Member shown dotted:
Max. − Web Stress = $\frac{1}{2} q l_1 l_2 \frac{\sec \alpha'}{p}$
Max. + Web Stress = $\frac{1}{2} q' l_1' l_2' \frac{r \sec \alpha'}{p}$
FIG. 3
FLOORBEAM REACTIONS
Intermediate Pier Reactions
Influence Line for R
Max. Intermediate Reaction:
$R = \frac{1}{2} q (l_1 + l_2)$
(l_1 and l_2 = The two Adjoining Spans)
FIG. 4
Equivalent Uniform Loads for Shears and Reactions ($l_1 = 0$)
FIG. 5
Max. Shear at C
*$S = \frac{1}{2} q l_2 - \frac{l_2}{l}$
Max. Reaction at A ($l_2 = l$)
$R = \frac{1}{2} q l$
l_2 | q
1000 | 6418
975 | 6429
950 | 6440
925 | 6451
900 | 6463
875 | 6476
850 | 6490
825 | 6504
800 | 6519
775 | 6535
750 | 6552
725 | 6571
700 | 6591
675 | 6612
650 | 6634
625 | 6658
600 | 6683
575 | 6711
550 | 6741
525 | 6775
500 | 6812
475 | 6852
450 | 6896
425 | 6945
400 | 7000
375 | 7060
350 | 7130
325 | 7205
300 | 7295
275 | 7400
250 | 7520
225 | 7660
200 | 7830
175 | 8035
160 | 8185
150 | 8300
140 | 8420
130 | 8540
120 | 8680
110 | 8840
100 | 9000
95 | 9060
90 | 9140
85 | 9230
80 | 9315
75 | 9410
70 | 9470
65 | 9600
60 | 9800
55 | 10100
50 | 10450
45 | 10900
40 | 11310
35 | 11830
30 | 12650
25 | 13625
20 | 15000
15 | 16000
10 | 18000
5 | 24000
* Note: In the formula for "S" the effect of wheel D is neglected. To correct for this, subtract $30{,}000 (1 - \frac{l_2 + 8}{l})$ from the above value of S
Curves of Equivalent Uniform Loads for Moments and Shears.
Plotted for Cooper's E 60 Loading. Values are given in pounds per lineal foot per track.
Copyright, 1915, by D. B. Steinman.

Logarithmic scales were used for the coördinates in order to avoid excessive disparity in the spacing of the curves. The interval between the curves is 100 lb. By simply reading the nearest curve, the maximum error is 50 lb., or less than 1%. The original chart,* three times as large as the accompanying reproduction, can be read by interpolation between the curves to the nearest 10 lb., yielding the final results accurate within about 0.1%.

The chart can also be applied to continuous trusses and other indeterminate structures. To find any stress we simply multiply its influence area by the equivalent uniform load given by the chart. The chart thus dispenses with the necessity of using moving wheel-loads.

Raising the Washed-Out Dos Rios Bridge

By F. W. Haselwood*

The Dos Rios highway bridge, which was partially wrecked by the sliding out of the piers at one end and the tilting of those at the other, was raised and replaced by a method different from those described in Engineering News of Feb. 18 and Mar. 11, 1915, pp. 352 and 495.

The bridge spans the Eel River at Dos Rios, Mendocino Co., Calif., immediately below the junction of the south and middle forks of the river. It is a camelback wooden truss 320 ft. long, supported at each end by cylindrical piers. At the east end the piers were shallow and the bridge appeared to be resting on the rock ledge. At the west end the piers were about 30 ft. high, with the footings resting on exposed rock.

Construction of the Northwestern Pacific R.R. compelled changing the west approach to a sharp curve, which left the piers without longitudinal brace. Under these circumstances the bridge was wrecked by undermining of the piers.

From the east end of the bridge, Mendocino County had partly completed a deep cut on the new Dos Rios-Covelo highway; failure of the road contractor or inspectors to care for the drainage resulted in a large flow of water down this cut to the bridge during the heavy storm of 1914, and what appeared to be rock ledge under the piers at the east end of the bridge washed out and slid toward the river, taking the piers with it. The east end settled 7 ft. and moved westerly 3.5 ft., this longitudinal movement tilting the west piers also. Bracing and cable ties were placed immediately to prevent further damage. The views in Fig. 1 show the bridge as it was before the work of raising began.

Among the proposed methods for replacing the bridge in its proper position were: (1) Raising it by jacks under the castings and pulling it into position by means of cables and turnbuckles; (2) raising and shifting at the same time with a gallows frame, with cables and tackle attached to the end pins, to be placed a short distance easterly from the proper position of the end of the bridge; (3) raising and shifting at the same time, with cables passing over a stiffleg or A-frame, or two of them, and anchored to deadmen and tightened with turnbuckles. When the débris that had washed over the end of the bridge was cleared away it was found that the end castings were both broken and the bridge was held in place more by a miracle than otherwise. Therefore, none of the plans mentioned for raising could be employed. Placing falsework under the entire structure was then considered. At the writer's suggestion a simpler method was adopted.

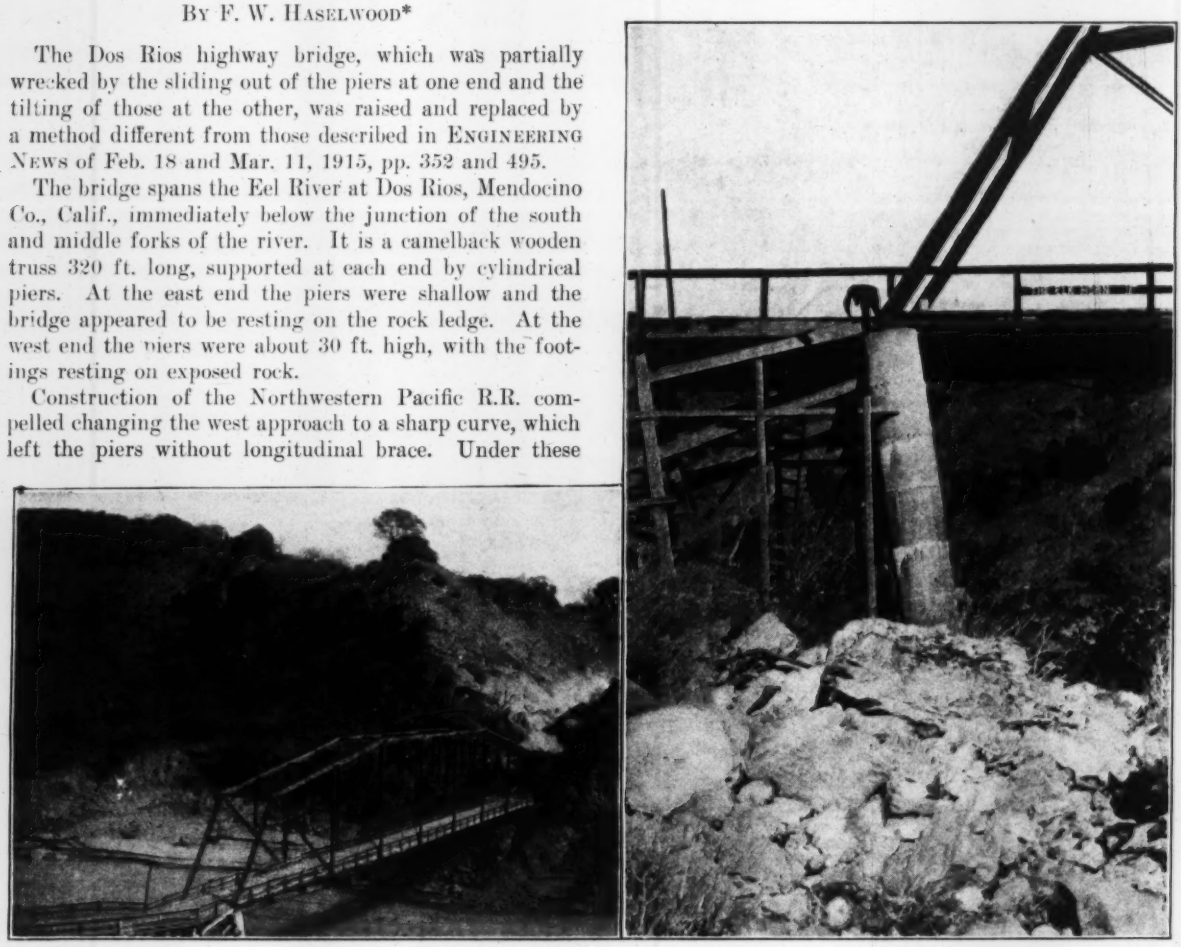


Fig. 1. Displaced Condition of Dos Rios Bridge over Eel River, California

*Copies of the full-size chart, 27 in. by 32 in., printed in black lines on linen, specially prepared for office use, complete with sketches and formulas, will be sent to any address by the author upon receipt of price ($1).

*Civil Engineer, Willits, Calif.

The bridge is divided into 8 main panels of 40 ft., subdivided into 16 subpanels of 20 ft. At the east end a false diagonal strut was placed in the second subpanel of each truss, and a false tie in the third, thus in effect shortening the bridge 20 ft.* Jacking falsework was placed under the new end-points, and the bridge was raised with hydraulic jacks. The total weight was about 160 tons. After the bridge was raised to the proper elevation and blocked up, it was thrust longitudinally into proper position by jacks placed horizontally at the west

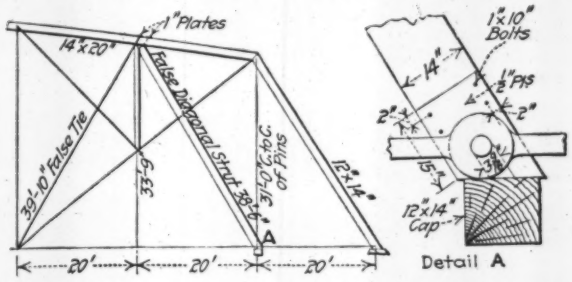


FIG. 2. SKETCH OF FALSE END-MEMBERS FOR JACKING DOS RIOS BRIDGE

end. New end-posts and castings were placed on the east end, a foundation was cleared to bedrock and a timber pier erected.

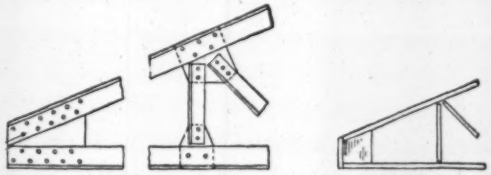
Fig. 2 sketches the general plan of placing the false members and a detail of the lower end of the diagonal strut. The floor-beam was removed to make room for the cap; it was not necessary to connect the horizontal bracing beneath the floor. The upper end of the strut was fastened to the top chord by plates over the chord and under the strut, connected by bolts. As the top chord was 20 in. wide and the false strut only 12 in., it was necessary to bore through the chord for these bolts. The rods of the false tie were hooked over the pin at the lower end and fastened with clamps. At the upper end they passed outside the chord and through a plate, which was held from slipping on the chord by spikes.

When the bridge was shifted easterly to its proper position, the west piers settled into place. Additional concrete placed at the footings remedied the damage done by the tilting.

It is proposed at some future time to replace the bridge with a steel structure on a grade 24 ft. higher than at present and with an extra span over the railroad.

NOTES

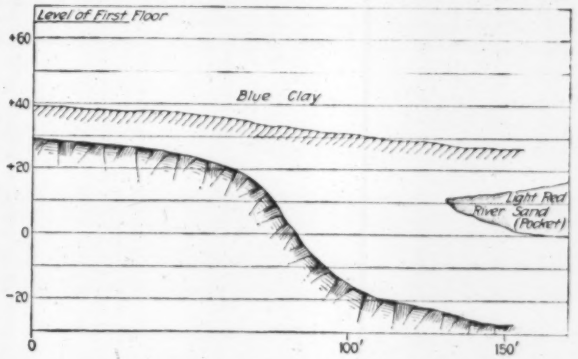
Chord Stresses in Eccentric Truss Connections—A correspondent inquires concerning the stresses existing at and near an eccentric heel connection of a truss, such as is sketched



in Fig. 1. Connections of this type are common. They are known to introduce bending moments in the chords. Although in ordinary practice these are neglected, they are often large and should be computed in those cases where connections of the type shown are unavoidable. The correspondent's inquiries relate to the amount of importance of the moments, a simple rule for determining their amounts, etc. The question is a little more complex when the chords are separated farther at the end, as sketched in Fig. 2. The rule should be applicable to both cases, of course.

*Compare with the method described, Mar. 18, 1915, p. 538.

Foundation Conditions, Merchants Bank, St. Paul—Unusually variable depth of rock affected the construction of foundations for the 16-story Merchants National Bank Bldg., Fourth and Robert St., St. Paul. The white sandstone which underlies the city, with top at about El. 72, city datum, is cut through by a ravine running back from the river in the center of the business district and filled with glacial drift capped with blue clay. The Pioneer Press Bldg., built 12 stories high



SUBSOIL SECTION AT MERCHANTS BANK SITE, REAR TO FRONT

about 1892, but later raised to 16 stories, is near the middle of the ravine. It has pile foundation. The site of the Merchants Bank Bldg., directly across the street, is on the shelving west slope of the ravine, as was found by borings. At the back of the lot rock is only 36 ft. below first-floor level, but along the Robert St. front its depth is 83 to 92 ft. The column foundations were constructed by sinking 6-ft. circular open wells with wood-stave lining to rock and filling them with concrete. Much water was encountered, requiring heavy pumping.

To Carry a Sewer beneath a Canal Embankment which rested on running sand at Rochdale, England, the invert of an existing culvert beneath the canal was replaced by a hollow cast-iron invert which has since served as a sewer. The culvert was of elliptical section, 8¾ ft. high by 7½ ft. wide, "formed of four half-brick rings" to quote from the London "Surveyor." The hollow cast-iron invert blocks were made of a special form, equal in sectional area to a 24-in. circular pipe. Each 2-ft. length of block weighed about 1400 lb. To give a solid foundation to the cast-iron invert sewer, as it may be termed, "finely ground liquid lias lime grout" under air pressures up to 60 lb. was forced into the sand through holes drilled in the brick invert of the culvert. After allowing time for the grout to set, the brick invert was cut out in lengths not exceeding 2½ ft. A few inches in depth of the grouted sand were then replaced by "fine cement concrete," shaped by means of a template and rammed down. The hollow invert blocks were then placed. The spaces between the iron block and the remaining portions of the brick invert were filled with a 1:1 stiff cement mortar and calked with an iron tool, as were also the V-shaped joints between the ends of the iron castings—the faces of the latter having been planed and fitted. The work was done under the direction of S. S. Platt, Borough Surveyor of Rochdale.

Asphalt Pavements on Old Brick Foundation—In Columbus, Ohio, we are using old brick pavements as a foundation for sheet asphalt on a number of light-traffic streets, and so far the practice has been satisfactory. Our method consists in raking out the joints between the old brick, concreting such holes as are worn through and using enough extra binder to bring the rest of the surface to a uniform crown. We use a brick gutter, made of new brick, these being raised enough for the outer edge to form a shoulder for the asphalt. This gives a slightly excessive crown at the brick gutter, but has the advantage of allowing us to maintain the original depth of the gutter at the curb line. We usually lay 1½ in. of binder and 1½ in. of sheet asphalt, with such extra binder as required, computed at the rate of 30 sq.yd. per cu.yd. box measurement. I do not consider an old brick foundation quite as satisfactory as concrete, but feel that the saving in the cost of this construction is enough to justify its use. I would not recommend it, however, for heavy traffic streets,

as the variable thickness of the binder might produce a wavy condition in the pavement.—Henry Maetzel, Chief Engineer, Department of Public Service, Columbus, Ohio.

Two 2400-Ton Hydraulic Jacks will be used at the Hell Gate arch bridge, New York, during erection of the arch steel by the cantilever method. The backstay is at present connected to the end of the top chord, but later attachment will be made by a second and higher backstay to the fifth panel-point. To transfer the load to the second backstay, two 2400-ton jacks are set under its saddle and the saddle jacked up until it takes the stress out of the first backstay, whereupon the latter will be disconnected. Then the saddle is to be blocked up, so that the load does not remain on the jacks. These jacks have 40-in. plungers. The jacks weigh about 14 tons each.

Weld Repair on Large Dredge Dipper—A dredge bucket on the "Grampus," the largest dredge working on the Florida East Coast Ry. extension at Key West, was broken, and attempts to weld it with oxyacetylene equipment failed. Therefore the old metal was cut away with the oxyacetylene

WELD REPAIR ON DREDGE DIPPER

flame and "Thermit" resorted to for filling in the two gaps left. One was 3 in. wide and 16 in. deep and the other 3 in. wide and 7 in. deep; 150 lb. of the alumino thermic compound was used on the large weld and 50 lb. on the smaller one. The pouring gate and riser were cut off with the oxyacetylene flame.—P. E. Rosselle, in a letter to Goldschmidt Thermit Co., New York.

A Road Oiler for Transporting Fuel Oil to supply steam shovels is an innovation of the Los Angeles County, Calif., Highway Department. In order to keep the shovels amply supplied with fuel a motor-truck road oiler, similar to the one described in "Engineering News," Dec. 10, 1914, p. 1176, was used to transport fuel oil direct from the oil fields to a storage tank near the site of the road work. A stream crossing where a bridge had not yet been constructed prevented the oil from being carried by the truck all the way to the shovels. The truck has an interior rotary pump designed to distribute asphalt in road surfacing work. This pump was utilized to force the fuel oil across the stream, a distance of 400 ft., through a 2-in. pipe against a pressure of 100 lb., to two storage tanks having a capacity of 10,000 gal. From the storage tanks the oil was carried to the steam shovels by small tank wagons drawn by mules. The steam shovels burned about 800 gal. of oil per day each.—Frank Reed, Los Angeles, Calif.

Engineering a Political Campaign—A correspondent in a New York State town sends us a blueprint poster, 11x18 in., of which the accompanying illustration is a reproduction. He gives the following explanation: "During my campaign I advertised by putting the inclosed blueprints in every business place throughout the town. These are original and are turned out of my office, and I wish I might send them to some other engineer looking to be elected to a public office. I feel that I have not only advertised myself but every other engineer in this locality."

P. S.—The posters elected him.

Longitudinal Joint vs. Longitudinal Cracks in Concrete Pavements—A study of concrete pavements constructed in Macoupin and Madison Counties, Illinois, shows that the percentage of longitudinal cracks increases as the width of the pavement slab increases. Longitudinal cracks have been entirely eliminated in one instance by the divided-slab method of construction, where a longitudinal joint is located along the center of the pavement. The condition of the surface of this pavement (High St., Carlinville, Ill.) is excellent. Some of the transverse joints were poorly constructed and show abrasion. The longitudinal joint shows much less wear, demonstrating that a protected longitudinal joint will stand the abrasion of traffic much better than transverse joints, and will wear much less than an unprotected irregular crack. I claim that the longitudinal joint answers the purpose of increased thickness in the case of wide slabs. Many miles of concrete pavement in Minnesota and Illinois having a width of from 8 to 10 ft. and a thickness of 6 in. show no longitudinal cracks or other defects. The least thickness sufficient to prevent such defects is the economical thickness. If, then, a concrete pavement wider than 10 or 12 ft. can be designed with a central protected longitudinal joint, the thickness of the pavement may safely be held to 6 in., the usual unsightly cracks avoided, and the cost of construction reduced from $1000 to $3000 per mile.—S. T. Moore, Consulting Engineer, Carlinville, Ill.

FIGS. 1 AND 2. LOS ANGELES COUNTY ROAD OILING TRUCK IN EMERGENCY USE, PUMPING FUEL OIL FOR STEAM-SHOVEL SUPPLY

Editorials

How a $200,000 Bridge Will Cost $800,000

Redeeming bonds for public improvements by new bond issues has been a favorite amusement in many American cities. Some cities are beginning to realize the costliness of that sort of play. Judging from statements in the *Pioneer-Press,* St. Paul is now one of the reformed cities, but a late convert. That journal recently said:

> Thirty years ago the city built the Robert Street bridge and issued $200,000 in bonds to pay for it. That was in the old days when we indulged in a perfect orgy of bonds. Since then we have paid $300,000 in interest on those bonds and not a dollar of the principal. But we got the bridge, though it has cost us $500,000 instead of $200,000.
>
> Now we must refund those bonds and it will cost us nearly $300,000 in interest, with the original $200,000 yet to take care of. So the Robert Street bridge will cost St. Paul about $800,000.
>
> And now, without a dollar paid on its cost, the Robert Street bridge is antiquated and unsuited to its purpose. It was a failure from the beginning and must be replaced by an adequate structure. St. Paul will have paid, when it pays it, $800,000 for a failure.
>
> Here is a kindergarten lesson in city bonds. Is it any wonder the taxpayers of St. Paul want to be shown before they give their consent to the issue of more bonds?

The sound way to finance such improvements is either to pay as you go or else to issue bonds which will not run longer than the useful life of the improvement. If bonds are issued it is far safer to pay them off year by year than to accumulate a sinking fund for possible diversion to other uses.

Deep-Sea Diving Experiments Yield Results

Engineers at all familiar with the ordinary limitations of the helmeted diver have no doubt been somewhat mystified at the press dispatches from Honolulu announcing the daily submersion of divers to depths below 250 ft. It has hitherto been assumed by the civil-engineering profession—which, to be sure, has not had very much to do with such deep diving—that the limits of human endurance were reached at a depth of 200 ft. and that anything deeper than 175 ft. was the performance of a superphysique rather than an ordinary human possibility.

The very interesting item on our news page—which comes through our Washington correspondent direct from an official of the Navy Department—indicates that a new method of deep-sea diving has been established through long and dangerous experimentation and that the Honolulu accident affords the first field demonstration of the practicability of the method. A depth of 288 ft. is reported to have been reached there without injury to the intrepid diver.

While the need of such deep diving will probably be confined to marine operations and particularly to the salvage of sunken vessels, the application to civil engineering practice is within the range of possibility, and all engineers interested in such work will do well to bear in mind the studies and tests conducted first by the British and later by our own navy.

After Sixteen Years

The long-looked-for final report of the British Royal Commission on Sewage Disposal has appeared. It proves to be a concise summary of the earlier reports of the Commission, and also of the ninth report. The latter is published simultaneously with the final report and deals with standards for trades-wastes effluents and with the disposal of domestic wastes in rural areas.

When the Royal Commission was created, in 1898, the Local Government Board for England and Wales was insisting on land treatment for sewage, regardless of whether suitable land was available or how efficient the so-called bacterial methods of treatment then coming to the front might be without land treatment. There was also a strong tendency at that time to demand that all sewage-works effluents measure up to some arbitrary standard, regardless of the character and uses of the stream which received the effluent.

For Great Britain the greatest service rendered by the Commission was its early declaration that land treatment need not always be insisted on, since artificial processes alone would produce a nonputrefiable effluent. Not only Great Britain, but also the United States and the rest of the world, is indebted to the Commission for its rational view of sewage disposal, which holds that each case should be decided according to local conditions and recognizes the fact that for the most part the proper object of sewage treatment is to prevent nuisance, not to convert sewage into drinking water.

The efforts of the Commission to establish provisional standards for ordinary and trades-wastes sewage-works effluents, subject to modification to suit local conditions, seem likely to be of practical value in England and to serve as useful suggestions here.

The reports urge repeatedly and with force the need of a central authority to control the pollution of streams and tidal waters in Great Britain. District or rivers authorities, under the jurisdiction of the proposed central authority, are also advised. At present two or three rivers authorities exist and the Local Government Board has a large measure of control of plans for new works and extensions to old—provided loans are to be obtained to pay construction costs; but it does not seem to have the power to order sewage-works built or to insist on proper operation after works are constructed.

No better illustration of the lack of power—or is it lack of courage, or sheer inertia?—of the Local Government Board in the control of sewage-works can be given than its waiting for the appointment of a Royal Commission to free British municipalities from their thraldom to the land treatment of sewage—imposed by a still earlier commission.

It remains to be seen whether England will profit more than it has already done from the 16-year task of its Royal Commission on Sewage Disposal. Further progress depends upon legislation creating the central authority recommended by the Commission and giving it ample powers. Some such legislation is certainly needed.

The Slides in the Culebra Cut on the Panama Canal

The newspapers have given so much space to the slides in Culebra Cut on the Panama Canal since it was open to traffic that the authoritative statement in this issue by W. G. Comber, who is in charge of the dredging work on these slides, will be read with especial interest. Culebra Cut is the hugest piece of excavation in the earth's crust ever attempted by man, and the slides on its slopes are probably bigger than any landslips with which man has ever attempted to keep pace. Of course there have been cases, such as the movement of a whole mountain side at Frank, Alberta, a few years ago, or the notable mountain falls in the Alps, or possibly the slides on the Fraser River due to the irrigation of farms on the steep slopes, where larger volumes of material have been set in motion by some human agency; but the digging away of the slides at Panama is the largest task in attempting to cope with the movement of a great landslip as it comes down that has ever been undertaken.

It seems hardly necessary to say in these pages that this is the only possible way of dealing with these slides. The forces producing them are too vast to be held in check by anything in the way of retaining walls; and an attempt to stop the slides by an elaborate system of surface drains to prevent water percolating to the underground strata was made by the French a generation ago and proved to be absolutely useless.

That the methods and machinery in use at Panama are capable of dealing with these huge slides will be clear to any engineer who reads Mr. Comber's paper. The area of hillside in motion is between 50 and 60 acres in the case of the Cucaracha slide and nearly as large an area in the slide at New Culebra, but the dredges working at the foot of these slides have dug away the material at a rate per month exceeding 350,000 cu.yd. in several months of the work on the Cucaracha slide, while on the New Culebra slide during the 28 days of February 573,846 cu.yd. was removed. As only about 5,000,000 cu.yd. is in motion in the New Culebra slide, it is evident that less than nine months' work would remove the whole slide were it all to move down into the canal prism.

There may be further movement of earth and rock into the canal prism at troublesome points along the Culebra Cut, as indicated in Mr. Comber's article, and there may be periods when traffic will for a short time be again interrupted. There is every reason to believe, however, that the slopes are approaching a condition of stable equilibrium, so that there should be comparatively little movement of material into the channel after that now in motion has been taken care of.

It seems well to emphasize these facts, because the newspaper reports concerning the slides have led certain timid souls to question whether somebody had not blundered in choosing the Panama route for an isthmian canal. Attempts have even been made to create a sensation out of the proposed purchase by the United States of the franchise for a canal across the Nicaragua Isthmus, ingenious press correspondents evolving the theory that this was being done because the Panama route would have to be abandoned on account of the slides! A still more curious idea, which seems to have obtained lodgment in the popular mind, is that the slides in the Culebra Cut are evidence that a mistake was made when a lock canal was chosen instead of a sea-level canal. Of course every engineer, as well as every layman who understands anything of the canal construction, realizes that the slides are a final and absolutely conclusive proof of the wisdom of the decision to build the canal with locks, reached in 1906 after one of the sharpest controversies that has ever agitated the engineering profession.

The summit level of the canal is 87 ft. above tidewater, and up to the present time nearly 30,000,000 cu.yd. of material has been excavated from the various slides in Culebra Cut. Of course, the deeper the channel is made, the steeper will be the slope of the material on the hillsides along the cut and the greater the amount of material which will slide into the channel. One can only wonder how many million cubic yards would have slid into the canal had the United States attempted the herculean task of digging the canal at sea level, as it came so near doing nine years ago.

Competition of a Government Bureau with Private Engineering Practice

Engineers the country over will read with great interest the account published in our correspondence column this week of a contract taken by the U. S. Geological Survey to make a topographic survey of Harris County, Texas, for which work 14 private engineering firms had previously submitted bids. A statement regarding the incident by the Director of the Geological Survey is also printed, so that our readers have both sides of the case presented to them and can form their own judgment.

The topographic work of the Geological Survey has had from its inception the hearty support of the engineering profession. Engineers more than any other class of men understand the value of accurate foundation topographic maps in connection with engineering work of many kinds. The Survey has had the strong support of engineers, therefore, in securing appropriations to continue year by year the extension of the topographic network and the publication of its standard maps based on these topographic surveys.

It will be news to the engineering profession, however, that the Geological Survey is now going beyond this foundation survey work and is undertaking detailed topographic surveys for specific engineering works. Of course, no private engineer can hope to compete with a Federal Government bureau. If there is any possible doubt upon this matter it will certainly be removed when it is considered that the Survey offers to do the work at cost, and in addition to contribute from its own funds so much of the cost as is represented by the work necessary for the foundation survey and map.

Where does the Geological Survey propose to draw the line in this field of work? Fourteen engineering firms went to the expense of making estimates and bids for the drainage survey of Harris County—by no means a trivial matter when it is understood that one of these firms had its representative make three special trips to Texas in an endeavor to secure the work. According to our information, the Geological Survey made its offer with full knowledge of these previous bids by private engineers. Does this establish a precedent for future work by Government

engineering bureaus? If so, the private engineer who offers to make a topographic survey for land drainage or water-supply, or any other public purpose, for a city or county must always run the risk of finding himself underbid on the work by a Government bureau.

It should be obvious that the Geological Survey cannot perform all surveys of this sort that must be made in all parts of the United States unless it is enormously to expand its present organization. Why, then, should it undertake this detail work at all? If the Survey is conferring a valuable benefit upon Harris County, Texas, by making this survey for it at a low cost, why is not the Survey equally bound to furnish the same advantage to any other county in any other state of the Union that may desire it?

The engineering profession throughout the United States has a high regard for the U. S. Geological Survey; but we believe there will be general agreement that the Survey is going outside of its proper field when it leaves the work of foundation topography and undertakes to make the detailed surveys required for any specific engineering work.

An Engineering Investigation of New York Highway Work

The Department of Efficiency and Economy of the State of New York (a department which the present legislature has abolished) has just published a report on the construction and maintenance of New York highways, prepared by an advisory board made up of Prof. Arthur H. Blanchard, of Columbia University, and Prevost Hubbard, Consulting Chemical Engineer, formerly connected with the Bureau of Road Inquiry.

As most of our readers are aware, New York highway work has been the subject of frequent and fruitful investigations during the past two or three years. Most of these, however, have been conducted by lawyers or politicians, or lawyer-politicians, who were chiefly concerned in finding out where fraud in highway work could be charged or proved. The investigation by Messrs. Blanchard and Hubbard was an investigation by engineers to find out how New York's road work could be improved. Their report is an expert criticism of New York highway work, literally from the top to the bottom.

Not all of the criticism is adverse. The board says, for example, that Commissioner John N. Carlisle deserves great credit for the reorganization of the department which he accomplished during his term of office. One has to turn over a good many pages of the book, however, to run across such isolated cases of commendation. Broadly speaking, the advisory board criticizes everything, beginning with the Constitution of the State of New York at one end of the scale and at the other end citing specific instances by the dozen where it is claimed the state's money has been wastefully expended in road repairs.

To take first of all a bird's-eye view of the whole New York highway situation, it seems well to correct the erroneous idea, very generally held, that the New York highway work has been one big carnival of graft and corruption and that most of the state money spent on highways has been stolen. There has been undoubtedly a certain amount of dishonesty in connection with road construction in various parts of the state, but it is probable that the work has suffered a great deal more from ignorance and incompetence and mistakes than it has from downright stealing.

What has happened in the New York highway work is exactly what might have been expected to happen. In 1905, with a great flourish of trumpets, a $50,000,000 state bond issue was voted to build a system of state roads. Of course, the public demanded that the roads should be built at once. If the work had been planned with a view to efficient performance, the first problem tackled would have been to create an efficient organization to carry on this great work and then to make the proper studies to determine what kind of roads should be built. But time for either of these tasks was lacking. The demand was that as much money as possible should be spent before a new state administration should come into power. There was more concern as to who should get the places in the organization than there was that the organization itself should be efficient and competent. There was more pulling and hauling to see that each rival kind of road material should get its share of the contracts than there was endeavor to select the type of road best suited to each locality.

We may add to this the fact that in 1905 very little was known about the effect of the automobile on improved highways and that it was impossible to oversee the enormous increase in automobile traffic and in the use of the roads by motor omnibuses and motor trucks. Add to this again that each county and town and village and city devoted all its political influence to get as much as possible of the state appropriation spent within its own borders. Is it any wonder, under all these conditions and others which might be enumerated, that the system of roads which New York has to show for the money expended is very far from ideal? It is not at all strange that Messrs. Blanchard and Hubbard can fill a book of 365 pages with criticisms of these roads.

In every art and industry, knowledge of mistakes is valuable; and this report should be in the library of every highway engineer. It contains a great deal of matter worth careful study, even though the reader dissents from the conclusions reached by the critics—and there are many cases where the dissent will be very radical.

For example, Messrs. Blanchard and Hubbard recommend the use of an asphalt filler instead of cement grout for brick pavements, and this in the face of the fact that the engineers who are most interested in the success of brick pavements are unanimous in their preference for cement grout. The critics also favor asphalt filler for stone-block pavements.

Again, they recommend a greater width for state highways. Among other methods of ascertaining how wide highways ought to be they wrote to all the American automobile clubs and associations asking them to state what they thought should be the minimum width of brick and bituminous pavements. Naturally, such organizations recommended a liberal width, as they are not interested in the taxpayer's point of view. The road engineer, however, is confronted with the fact that the wider he makes his road, the more it costs to build and to maintain. Some state-highway departments are building single-track roads, only 9 or 10 ft. wide, on routes of moderate traffic.

A considerable part of the advisory board's report is devoted to criticisms of the type of roads selected for use in specific localities. In general the board charges that waterbound macadam has been adopted on certain roads

of heavy traffic, and on the other hand that a large amount of money has been expended in laying expensive brick pavements on short sections of state highways where the road passes through small villages. It may be said in comment upon this latter charge that there is no doubt that a large amount of the state's highway appropriation has been expended to benefit particular localities rather than the state as a whole, and especially in paving city and village streets. This, however, is a general condition which obtains in a large part of state and county highway work. We know of no state in which the highway engineer is given such supreme authority that he can apportion the state money for road construction where it will do the most good to the greatest number regardless of the demands of specific localities.

Further than this, a pretty strong defense can be made of the practice of laying a permanent roadway, such as brick pavement, where a state road passes through a town or village. There is always a great deal more traffic within the limits of even a small-sized village than on the main highway. There is also good reason for laying along a village street a dustless pavement which will require comparatively few repairs that does not hold where the road runs through open country.

A large part of the report is devoted to the discussion of specifications for asphalts and tars used in various kinds of bituminous macadam pavement. It goes without saying that these specifications have long been, and still are, the subject of a very sharp controversy, and there will doubtless be many who will dissent from the conclusions of Messrs. Blanchard and Hubbard. Into these troubled waters we do not intend to enter. It does seem proper to suggest, however, that the report before us might have been more valuable had its authors given more space to discussion of the advantages of concrete and brick roadways in comparison with the various types of bituminous macadam. It is of course true that the reputation of concrete roads has suffered in New York because of the badly designed and badly built concrete roads within its borders. The rapid increase in the use of concrete roads and brick roads in the Middle West and on the Pacific Coast, however, cannot be ignored in the East.

It is well also to frankly admit that nobody in the highway-engineering field, from top to bottom, is above the possibility of making mistakes, and very serious mistakes, in road construction. It could not well be otherwise with the rapid changes in highway traffic and in road materials and methods and in the increasing experience of both engineers and contractors in the handling and assembling of these materials. There is room, therefore, for a very large charity in dealing with the work which has been done in road construction in New York.

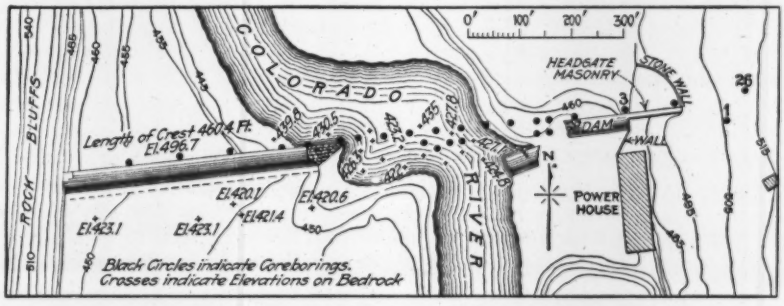
Letters to the Editor

Caverns under the Austin, Texas, Dam

Sir—I was much interested in the article in ENGINEERING NEWS of Mar. 18, 1915, p. 528, on the reconstruction of the Austin, Texas, Dam, and especially that section referring to the reticence and mystery surrounding this very important project. Inasmuch as the Water and Light Commission gave the fullest publicity to the very advantageous contract of 1912, by which the contractors undertook the sole liability for the permanency of the structure, it would seem that the commission at least should be absolved from the implication of secretiveness. At the same time, your readers may be interested in a brief reference to some of the events leading up to the consummation of this contract by which the city of Austin secures the reconstruction of its dam and power plant.

On Apr. 16, 1908, Walter G. Kirkpatrick, Consulting Engineer, then of Jackson, Miss., now of Birmingham, Ala., summoned the writer from the Shoshone Dam, Wyoming, and, after conferring with the Water and Light Commission of Austin, authorized the writer to proceed with diamond-drill coreboring at the site of the old dam to determine definitely the stratification underlying the vicinity.

This work was carried out partly on land and partly from a scow on which the drill was mounted to work in the stream. The greatest depth of any hole was at No. 26, as shown on the accompanying map, which reached a depth of 230 ft. from El. 513.71 to El. 283.71, or 212.99 ft. below the crest of the old dam, which was 66 ft. in height, thus making a test 146.99 ft. below the base of the dam. On the west side of the river a boring of 150 ft. in depth (from El. 437.8, or 142.9 ft. below the base of the old dam), together with numerous borings of lesser depth, as shown, demonstrated indisputably the very undesirable conditions existing at this site. Opposite test hole No. 3 a 5-ft. shaft was sunk to a depth of 37 ft. from El. 453.66, striking a subterranean passage.



MAP SHOWING BORINGS AT AUSTIN DAM

On June 6 an immense volume of water, carrying long-decayed drift, welled up in the center of the stream immediately above the line of the old dam and caused a rise of several feet in a few minutes. This phenomenon

impelled the writer to search for its cause. Learning of the existence of an open shaft located in Walsh's old quarry, about a thousand feet distant, on July 7, accompanied by members of the staff of the Austin *Statesman,* he entered this shaft and made a survey of the existing cavern. This survey showed that the cavern extended in a southwesterly direction nearly to the site of the dam, and an aneroid reading showed El. 420, at which point the cavity was of such width and depth as to make its further exploration impracticable without additional facilities. Falling water could be distinctly heard, and the inference was that it was finding its way in from the river.

The stratification disclosed by the core borings was of alternate moderately hard limestone, soft adobe and cavities.

The writer disclosed these conditions to the mayor and other members of the Water and Light Commission and to Mr. Kirkpatrick. A public meeting was held, at which the mayor explained the situation and justified his refusal to sign the ordinance then before the council. In this ordinance the city was obligated, in the event that construction did not proceed, to forfeit $700,000 of convertible bonds to a construction company then having a proposal before the Council. This refusal of the mayor to yield to the popular clamor for the reconstruction of the dam at any risk raised quite a furore locally; but he stood his ground, and the succeeding commission justified his action by securing the present contract, which eliminates the element of financial risk to the city.

Following this public meeting, Mayor Maddox, at the request of Mr. Kirkpatrick, secured the presence of Arthur P. Davis, Chief Engineer, U. S. Reclamation Service, and Louis C. Hill, then Project Engineer, Salt River Project, to confer with Mr. Kirkpatrick on the advisability and cost of reconstructing the dam at the old site. Assisted by Professor Symonds, Geologist, University of Texas, an exhaustive study was made of the borings, the site and the general surroundings. It was the unanimous conclusion that the reconstruction at this site was a risk the city should not assume, but in the event that it did so, the cost would range from $900,000 to $1,200,000, and that the plan of construction should be along the lines now being carried out at Austin. This conference ended on Oct. 17, 1908. The matter virtually rested there until January, 1912, when the present contract, putting all responsibility on the contractor, was formulated.

A. Cole Blanton.

Charlotte, N. C., Mar. 30, 1915.

Unfair Competition of the U. S. Government with Private Engineers

Sir—It is my desire to make formal protest through the columns of Engineering News against any engineering bureau of the United States Government entering into *unfair* competition with engineers in private practice.

While the prestige which any Government engineering bureau would naturally bring to an undertaking may always render its competition unfair, the circumstances which cause me to make this protest are so aggravating as to need added emphasis.

These are the facts: Late last summer the Commissioners of the Court of Harris County, Texas (W. H. Ward is County Judge), appropriated the sum of $35,000 for a topographic survey of the county, a drainage report and drainage plans. No further action was taken for some time.

In December the writer, who is associated as Geodetic Engineer with the firm of Brown & Clarkson, Topographic Engineers, Washington, D. C., made a special visit to Houston (the county seat) and in an interview with Judge Ward advocated (as had already been done by letter) that the work of making this topographic survey be done by contract; that specifications be prepared and bids be asked.

It was finally decided to do this, and in January bids were advertised in at least one Harris County paper, the Houston *Daily Post.* Bids were opened Feb. 10. Each was accompanied by a certified check for $1000, and it was specified that "parties bidding for the work . . . must familiarize themselves with the territory to be covered."

I met W. N. Brown, of Brown & Clarkson, in Houston about one week before the date set for the opening of the bids, and with him made a careful study of the topographic, climatological and forestry conditions of Harris County.

On Feb. 10 the county commissioners received and opened 14 bids for making a topographic survey, drainage report and plans as advertised. After the bids had been read they were held over until the afternoon of Feb. 12 for tabulation and study.

On the afternoon of that day the bidders were informed that on the preceding day a telegram had been received from the U. S. Geological Survey offering to make the survey, an offer which, it was said, might require some time to investigate. Judge Ward further said that as he was confident that all the bids received had been made in good faith, and as $14,000 was too much money to keep tied up indefinitely in present times, it was decided to return the certified checks to the bidders; but it was decided to hold the bids until a decision was made.

It must be said here that I am satisfied that the county officials had no knowledge of the U. S. Geological Survey's desire to do this work until after the bids had been received and opened. Rather desultory negotiations had been passing, I am told, between the officials of the Survey and the Houston Chamber of Commerce. These negotiations must have been secret in nature, since the county officials knew nothing of them.

In Washington, a few days prior to leaving for Houston, Mr. Brown saw several of the leading officials of the Survey without hearing anything about these negotiations, although he told them, in a friendly way, of his contemplated trip. This was not an unnatural thing to do, as only a little over two years ago the Survey refused to render any assistance whatever to the City of Cincinnati in making its topographic survey, on the ground that private engineers could be had to do the work, and that even to loan planetables to the city would constitute competition with private engineers, which was against the policy of the Survey.

Late in February Judge Ward visited Washington for a conference with the Survey officials, but was apparently unable to reach an agreement with them. In the opinion of Judge Ward, of his county engineer, and of

all engineers with whom I conversed, a topographic map showing 1-ft. contours is needed for making workable drainage plans of Harris County. An engineer not familiar with Harris County will hardly appreciate this until he learns that much of the prairie country around Houston has a slope of less than 2 ft. per mile.

There is in Texas a State Reclamation Department with headquarters at Austin, which has control over drainage matters in the state, and with which the U. S. Geological Survey is now required to coöperate in making drainage surveys. This state bureau has adopted for its maps a contour interval of 2 ft. and a scale of 1000 ft. to the inch. All topographic surveys made by the U. S. Geological Survey in coöperation with the Texas State Reclamation Department must then be under the supervision of that state bureau and in accordance with its own specifications. The foregoing is necessary to a complete understanding of what follows.

On Mar. 27 officials of the U. S. Geological Survey visited Houston, and on the 31st the county authorities accepted their proposition to make a topographic survey of Harris County.

While this survey is to be used primarily for drainage purposes, it is *not* to be made under the supervision of the State Reclamation Department. I understand that the contour interval is to be 5 ft., with intermediate contours "sketched in." The U. S. Geological Survey is to contribute a bonus of $5000 to the work, free use of necessary instruments, expert supervision, and furlough for the necessary topographers. The estimated cost of this survey is about $80,000.

So strong is the prestige of the U. S. Government surveyor (and it should be so) that an offer by a private firm of engineers to make this survey at a guaranteed saving of $15,000 under the estimate made by the U. S. Geological Survey met with no response. Said the county commissioners: "If you make this survey, we will have to hire some one to check you up; if the Government makes it, it will be all right."

I insist that the making of a county topographic survey and drainage plans is not a matter for a national bureau to take up. This is evidenced by the action of the Harris County authorities in asking bids for such a survey, and in the action of 14 engineers or engineering firms responding thereto. I protest against the U. S. Geological Survey's entering this field at all, and insist that its method of offering bonuses to be allowed to do this work is *most unfair* in all its phases. Furthermore, it was known to officials of the Survey that private engineers were making efforts to secure this work, while these same officials worked in secret with the Houston Chamber of Commerce.

I most respectfully request that ENGINEERING NEWS lend its large influence toward determining the justice of this protest, and how far, in future similar cases, the private engineer may expect this kind of competition.

HUGH C. MITCHELL,
Geodetic Engineer.

Washington, D. C., Apr. 8, 1915.

[A proof of the foregoing letter was submitted to the Director of the U. S. Geological Survey and his comment is as follows—EDITOR.]

Sir—The following facts are pertinent to the matter discussed by Mr. Mitchell.

The Houston Chamber of Commerce in 1912 and again in 1914 brought the matter of the topographic survey of Harris County to the attention of their representative in Congress, urging coöperation between the county and the Federal Government. Hon. John M. Moore, then representing the 8th District transmitted this request to the Geological Survey July 30, 1912, and on Nov. 5, 1914, his successor, Hon. Joe H. Eagle, renewed the request in which the appropriation by the county was specifically mentioned.

The official correspondence thus initiated could at no time be termed "negotiations, secret in nature."

Both in 1912 and the past year the policy of the Survey was explained to the congressman and the basis of coöperation stated as being that of proportional benefits, and in the case of detailed survey such as is desired and needed in Harris County, the contribution by the United States Geological Survey being an amount directly related to the cost of making surveys on the regular standard mile scale. In the discussion of the matter with Representative Eagle, the question was discussed of the ability of the Survey to find any available funds during the present fiscal year and the tentative decision reached in February was that $5000 might be contributed on the part of the Geological Survey.

I am advised that the bids received by the county ranged from something like $40,000 to $190,000, and the supplemental offer by a private firm of engineers mentioned by Mr. Mitchell, namely, at $15,000 under the Geological Survey estimate of $80,000, is in fact also at least $50,000 under the formal bid by the same firm.

The coöperation arranged in Harris County differs in no respect from coöperation carried on in 1909 between the Geological Survey and the Fifth Louisiana Levee District except in the sketching of the intermediate contours, and the fact that the proportion of the expense contributed by the Federal Survey is considerably less under the Harris County agreement. In neither instance nor in any other of the many cases of coöperation has the contribution on the part of the Federal Government been considered a "bonus."

The refusal of coöperation in the case of the survey of the City of Cincinnati was predicated upon the small area (approximately 100 sq.mi.) a map of which on the usual scale had already been published by the U. S. Geological Survey, which is in direct contrast with the 1700 sq.mi. of unmapped area constituting Harris County; and in all other cases of proposed coöperation the determining factor has been the Federal interest in the work. It is only in public work of this type that the United States Geological Survey participates under the appropriation "for topographic surveys in various parts of the United States."

GEO. OTIS SMITH,
Director, U. S. Geological Survey.

Washington, D. C., Apr. 14, 1915.

More Reinforced-Concrete Cantilever Bridges

Sir—In ENGINEERING NEWS of Mar. 25, 1915, p. 578, under the heading "A Balanced Cantilever Reinforced-Concrete Bridge," Henry H. Quimby describes a structure which, he says, "is believed to be a new type."

In 1904 the writer was confronted with a problem similar to that which faced Mr. Quimby, and he solved it in

a similar manner. At Bridgeport, Conn., a handsome masonry bridge, preferably of the arch type, was desired over Ash Creek, but no arch thrust was permissible, as the abutments had to stand on pile foundations in deep mud. A reinforced-concrete cantilever bridge was therefore con-

FIG. 1. HUSBAND'S FORD BRIDGE, HARFORD COUNTY, MARYLAND
(A cantilever resembling an arch)

structed in the form of a five-centered arch of 40-ft. clear span and 7-ft. rise.

The reinforcement was cold-twisted steel rods for tension stresses only. No steel was used in compression, the concrete being relied on to take all compression stresses. The bridge was so designed that under concentrated loads the middle portion of the arch might act as a beam or as a short span between the cantilevers.

Mr. Quimby remarks regarding the Chester bridge: "It would seem from the experience here that the joint with play for movement was an unnecessary refinement in the design and might have been omitted." The experience at Ash Creek confirms that opinion. No transverse joints were made in that bridge and no cracks have appeared.

ALBERT B. HILL.

New Haven, Conn., Mar. 29, 1915.

Sir—In 1911 the writer designed a reinforced-concrete cantilever bridge in the form of an arch, which it is believed antedates the one described by Mr. Quimby. This bridge* was built for the Maryland State Road Commission at Husband's Ford over Deer Creek, in Harford County. It is approximately 20 ft. wide and has a central span of 100 ft., two piers of 10-ft. width each and two balancing cantilever spans of 45 ft. each, making the total length of the bridge 210 ft. (Fig. 1).

This bridge antedates Mr. Quimby's bridge by several years, and the writer believes that he may be pardoned for saying that it is a somewhat bolder design on account of the fact that rods were used for reinforcement instead of built-up steel members, and also because the main span is 100 ft. clear, as compared with the 95-ft. span in Mr. Quimby's design. Furthermore, the shore cantilevers are 45 ft., as against 31 and 34 ft. for Mr. Quimby's bridge; and while the ribs of Mr. Quimby's bridge are spaced approximately 6 ft. on centers, the two ribs on the Husband's Ford bridge carry approximately 10 ft. of roadway.

Mr. Quimby is right in stating that this type of bridge is economical; this was proved in the case of the Husband's Ford bridge, the cost of which was approximately two-thirds of the cost of the arch bridge.

This type of bridge was also used by the writer for the

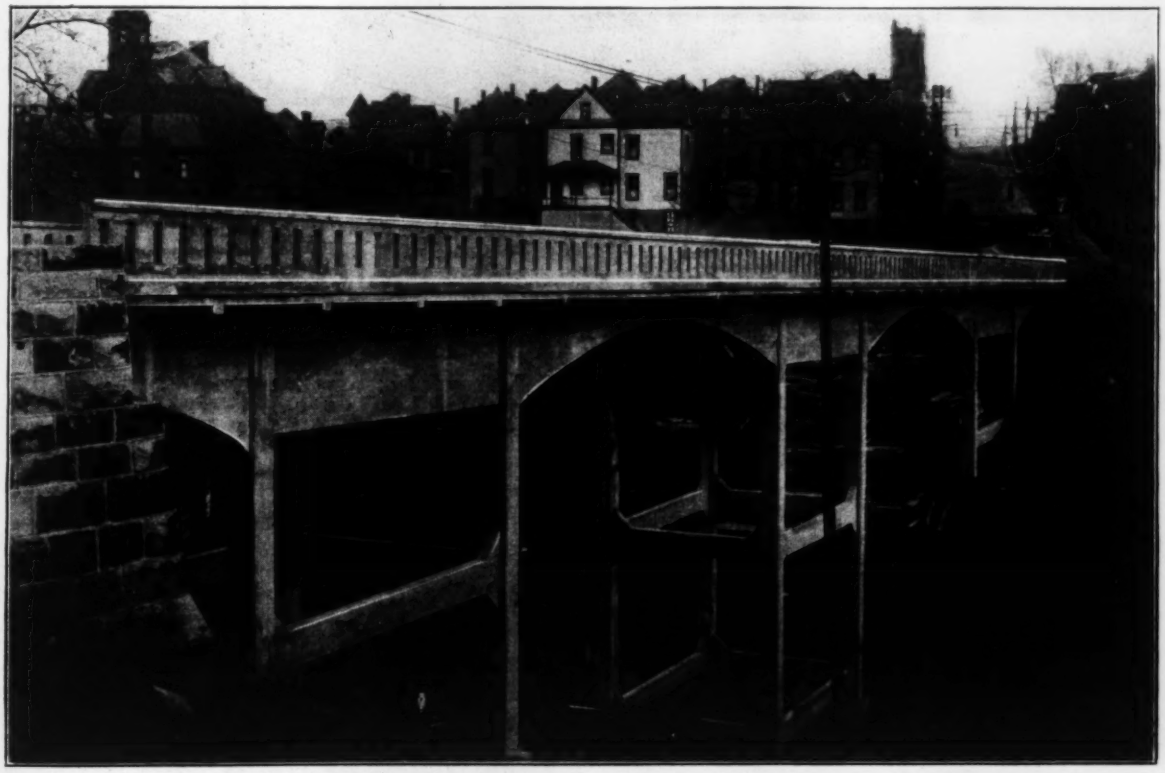


FIG. 2. FOURTH ST. VIADUCT, FAIRMOUNT, W. VA.; A CANTILEVER BRIDGE

*Described in "Engineering Record," Dec. 30, 1911.

Fourth St. Viaduct, Fairmont, W. Va. (Fig. 2), where, on account of the fact that the city wished to build two abutments by day labor, the viaduct is designed and constructed so as not to rest on the abutments at all, the shore spans being cantilevers and the abutments being used merely to hold back the earthfill at each end.

LAYTON F. SMITH.

513 N. Charles St., Baltimore, Md.,
Mar. 27, 1915.

Sir—The writer would like to put on record two bridges designed by him on the same principle as that noted by Mr. Quimby. The smaller one was built in New Zealand in 1909 and the larger one, over the Sweetwater River in San Diego County, Calif., was completed last year.

Both were designed for a uniform distributed live-load of 150 lb. per sq.ft. (equivalent to a 20-ton roller) plus 40% for impact. The San Diego bridge figured considerably cheaper than either an arch or girder design at the same site, principally because of the unsatisfactory nature of the foundation material. The New Zealand bridge was practically identical in estimated cost with a two-ribbed hingeless-arch design prepared as an alternative.

These two designs were based on the same assumption as that used by Mr. Quimby, that the dead-load balances at the middle of each pier with equal moments on either side, the maximum eccentricity of load occurring when live-load is concentrated at either end, over the counterweights—no live-load being on the span.

Mr. Quimby's conclusion that it is safe and desirable to omit the transverse joint in the floor is of interest, because in the designs noted the cantilevers were considered as being rigidly connected by a suspended span, there being no joint. In practice this assumption has proved to be sound. While it is probably true that in this design there is more or less arching action, there is also, in addition to the arch and balanced cantilever, a well-defined continuous beam action in the San Diego bridge. Probably this arch action makes for extra rigidity, which Mr. Quimby noticed in the bridge described by him. It would appear good design to omit the joint, thereby obviating the effects of the rocking action set up by the movement of the center of gravity due to moving live-load from one side of a pier to the other.

The larger bridge mentioned was built for San Diego County. George Butler, County Surveyor, and Paul Watson, Chief Deputy Surveyor, were in charge of design and construction.

F. H. FRANKLAND,
Bridge Engineer.

Lake Charles, La., Apr. 1, 1915.

NOTES AND QUERIES

The article in "Engineering News" of Apr. 15, 1915, on "Booster Pumps in Roseland Station, Chicago," should have stated that the pumps were designed and built by the Henry R. Worthington Works of the International Steam Pump Co., New York City. The figures of performance in shop tests of other screw-impeller units apply to runs made in the Worthington shops.

Some Substructure Details of the New Memphis Bridge

From a statement given out to various technical papers by M. B. Case, Resident Engineer, Arkansas & Memphis Railway Bridge & Terminal Co., the following items describing certain parts of the substructure work of the new Memphis cantilever bridge have been taken:*

The substructure, in addition to an anchor pier high up on the Tennessee bluffs, comprises five river piers varying from 25x46 to 42x90 ft. in plan, and varying in height above water from 51 to 81 ft. Each pier is founded on hard blue clay which is reached through a

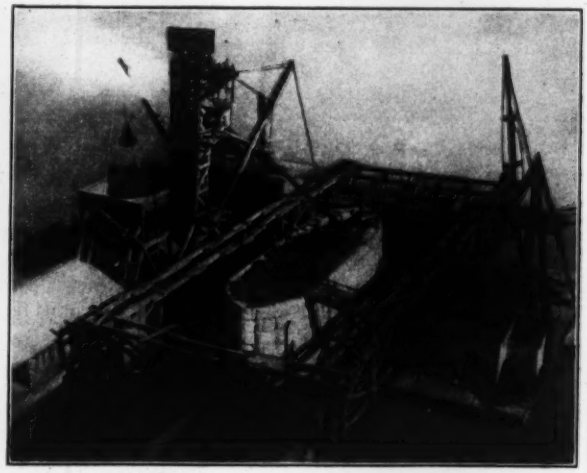


FIG. 1. FLOATING CONCRETING PLANT AT PIER OF THE NEW MEMPHIS BRIDGE
(Note dock around pier)

variable clay and a bed of sand. From the bottom of the foundation to the coping the highest pier is 195 ft. The construction is uniformly cut-stone faces, with concrete filling behind the stonework as forms.

CONSTRUCTION BARGES

The substructure was conducted with the aid of construction barges, 21 of which were built. Some of these carried the various equipment needed in the construction, and the rest were used as cargo boats. On the construction barges were erected air-compressor plants, concrete-mixing and delivering plants, piledrivers, derricks and mat-weaving outfits.

The concreting plant, on a 36x100-ft. barge, was of the familiar tower-and-chute type (Fig. 1). The material, washed river sand and screened gravel, was brought to the barge on 275-yd. scows and raised into hoppers over the mixer by clamshell buckets and derricks located on the concreting barge. Cement was elevated in a measuring hopper in a small car pulled up an inclined track by means of a line leading to a spool on the tower hoisting-engine. The cement-car ran back onto a track extension

*The new Memphis cantilever bridge (described in "Engineering News," Nov. 6, 1913) is being erected quite close to the well-known Memphis bridge built a number of years ago. The new bridge, beginning at the Memphis end, has an anchor arm of 186 ft. 3 in., a cantilever span of 790 ft. 5¼ in., a fixed anchor span of 621 ft., a cantilever span of 604 ft. and a deck approach span of 345 ft. Beyond this point on the Arkansas side a tower-and-girder viaduct continues over the bottomland for a distance of 2363 ft. to the west abutment. On the Memphis side the bridge is approached by earth embankments. The bridge will carry two tracks 14 ft. c. to c. between the trusses (34 ft. 6 in. c. to c.) and two 14-ft. roadways on brackets outside the trusses.

under the roof of the cement-barge, which was moored below the mixing plant, thus reducing the handling of the cement and the loss and damage of tracks during wet weather. The concrete-mixing barge was equipped with a 60-hp. locomotive boiler which furnished steam to the mixer engine and to the tower hoist, and also for the heating of the concreting material which was required only at infrequent cold spells.

The compressor barge for the pneumatic-caisson foundation work carried four locomotive boilers with a combined capacity of 300 hp. and with the necessary pumps and feed-water heaters. Three low-pressure compressors, with a capacity of 2500 cu.ft. of free air per minute, furnished air for the caissons, and one smaller machine high-pressure air for the boring tools and hammers used on timber construction. A small generator supplied current for light and for an elevator in the main shaft of the two deeper piers. The hospital lock was also on this barge.

River-Pier Construction

All of the river piers were built under compressed air in timber-frame caissons built on a pontoon for a part of their height and launched therefrom into the river, where the final height was added. On three sides of the pier site previous to the launching of the caisson a pile dock was driven (Fig. 1). This was used to hold the caisson in position until well landed and also served during construction as a working staging.

River-Bed Protection

Because of the swifter currents and the consequent tendency to scour, the bed of the river at Pier 2 was prepared to receive the caisson by sinking a willow-brush mat 200x300 ft. at the site of the pier. After the piles for the dock were driven through this mat, 29,000 burlap sacks filled with sand were deposited at the pier site to reduce the depth of the water about 8 ft. and to give an even bearing on which to land the caisson.

Willow mats extending 150 ft. either side of the bridge axis were afterward sunk and covered with riprap for

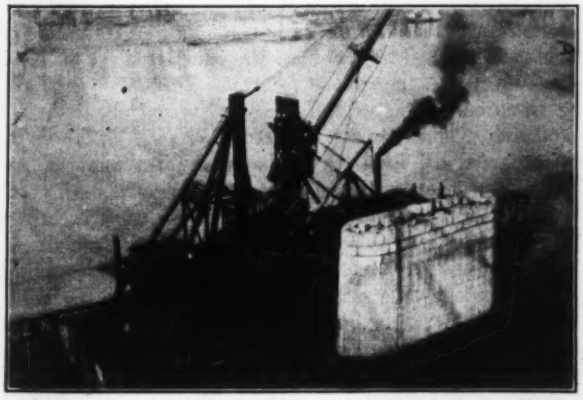


Fig. 2. Placing Stone on Pier from Construction Barge

the entire distance between Piers 2 and 3 to protect the erection falsework from scour during superstructure work. A mat was also placed along the Arkansas bank, extending 1000 ft. above the bridge, as a protection to that bank.

Approach-Pedestal Construction

The concrete pedestals shown in Fig. 3 are to carry the columns for the west approach. Each contains 53 cu.yd. of concrete and rests on a 13x13-ft. footing block, which in turn is carried on 16 reinforced-concrete precast piles. The concreting of these pedestals was rapidly done from a car running on a central track between them. The car is equipped with a regulation tower and spout delivery,

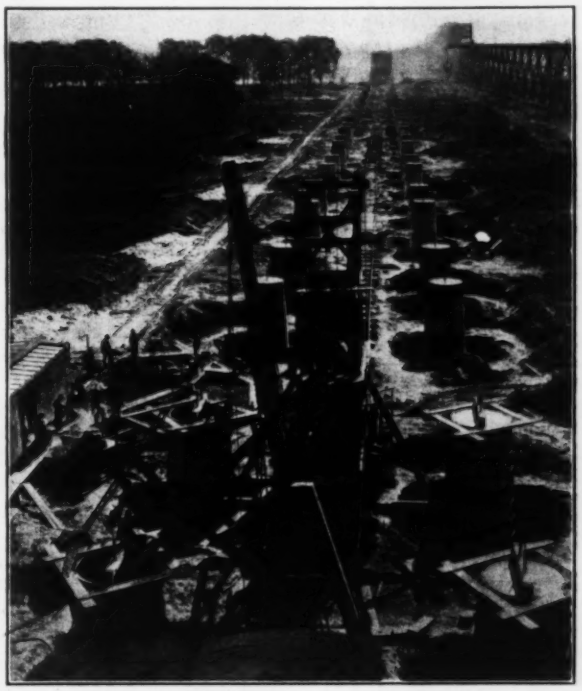


Fig. 3. Building the Concrete Pedestals for the Viaduct Approach on the Arkansas Side of the Memphis Bridge

and the four pedestals of each tower bent could be poured at one position of the plant.

All the work described was done between June, 1913, and January, 1915. Active preparation is now under way for the erection of the superstructure. The bridge is under the general direction of Ralph Modjeski, Consulting Engineer, with W. E. Angier, Assistant Chief Engineer. The substructure work was under contract to the Union Bridge Construction Co., of Kansas City. M. B. Case is the Resident Engineer representing Mr. Modjeski.

Financing Public Works in Meadville, Penn.—When it was first proposed to issue bonds, bankers advised that the issue be made at 5% for 30 years instead of 4% as had been done before, and predicted that the bonds could not be sold at 4%. But the City Council decided to issue 4's, and bids for the bonds were asked for on this basis. Quite a few bids were received, some at a fair premium and some at par, but not enough bids were received to take care of the entire issue. The Council then decided to offer the whole issue at par, and in a remarkably short time the whole issue was taken care of and oversubscribed about $20,000. All the bonds were sold locally in denominations of $500, none of them being taken by the banks. An amusing feature was the fact that $1500 was taken by an old German in the employ of the street-cleaning department, who was very much worried lest someone should find out that he had been buying bonds and he should lose his job.—B. F. Miller, Jr., City Engineer, of Meadville, Penn.

News of the Engineering World

Failure of Lyman Dam, Arizona

The Lyman Dam was an earth dike across the main channel of the Little Colorado River about 12 miles south of St. Johns, the county seat of Apache County, Arizona. It was completed August, 1913, and during the years 1913 and 1914 the reservoir was allowed to fill to a height of 39 ft. The dike was 450 ft. long on bottom, 840 ft. long on crest, 65 ft. high, with a 12-ft. crest and a slope of 2 : 1 on both sides; it contained 200,000 cu.yd. of material. It was faced on the upstream side with 6 in. of gravel and 12 in. of hand-layed riprap. The dike was made of the finest quality clay and had a puddled-clay core wall 12 ft. wide, and the bond between the dike and side of reservoir was carefully made with corrugations 2 ft. deep, 2 ft. wide and 3 ft. apart, tamped with choice clay. The reservoir had a capacity of about 40,000 acre-ft., being next in size in Arizona to the Roosevelt reservoir. There was a natural wasteway 117 ft. wide, situated about one-quarter mile from the dike. At the time of the break, the water depth at the dam was 53 ft. and the reservoir contained 29,000 acre-feet.

The Lyman Irrigation Project embraced an area of 15,000 acres immediately adjacent to the town of St. Johns, of which 6000 acres was in an advanced state of cultivation, the water-supply being obtained from the Lyman Reservoir. The cost of the project, including the dike, was approximately $200,000.

Owing to the topography of the country between the dam and the irrigable land, it was necessary to place the outlet conduit at the 20-ft. contour, which left 20 ft. of dead water. The outlet and penstock were constructed of reinforced concrete placed under most careful supervision and having two 30-in. Ludlow valves.

The break occurred at 10:45 p.m., Apr. 14, just after the dike had been carefully inspected by the superintendent, when it showed no signs of weakness. During the day of the break, the directors of the company, accompanied by Field Examiner R. G. Mead of the U. S. General Land Office, went over the entire project and visited the dike, at which time no evidence of cracking, settling or seepage could be detected.

The first evidence of the break was a sudden rush of water coming out at the base of the dike in the center of the channel, which was instantly followed by a volume of water rising to a depth of 30 ft. on the lower side of dam. The water immediately cut out a section forming an arch with an estimated span of 75 ft., which collapsed as the cutting increased, leaving a gap 350 ft. in length. The penstock and conduit were undamaged.

The dike had been constructed in two sections, the first section being over three-fourths the length of the completed dike and on the north side of the channel. The remaining section was then constructed across the channel while the river was at low water. During the construction of the last section, the water rose against the dike as the construction work progressed, a 3-ft. freeboard being maintained until the outlet conduit was reached. The accepted theory of the failure is that the portion of the dike across channel below the outlet conduit did not have an opportunity to dry out and properly settle; consequently, this section was materially weak and gave way when the pressure became too great. As the portion of the dike across the channel was built against an increasing pressure of water it was no doubt saturated to the core wall and above the outlet conduit by means of capillary attraction, which destroyed the bond.

It is estimated that the section destroyed can be replaced, the canal repaired and an outlet conduit placed at the bottom of the dike at a cost of about $75,000, which would mean an increased cost to the project of $5 per acre, making the total cost of $17.50 per acre, which is considerably less than any project of its size. The damage to property, highway bridges and other projects below the dam amounts to over $300,000. [Telegraphic dispatch from W. C. NICHOLS, St. Johns, Ariz.]

Steel Frame Blown Down during Erection

Part of a steel frame of Building No. 75, being erected for the Willys-Overland Co., Toledo, Ohio, collapsed during a heavy gale on Apr. 10, killing one laborer who was on the building and injuring several others. The building is 200 ft. wide, 1010 ft. long, and three stories in height. The second floor and roof are supported by steel framing, the columns of which extend without splices from footings below the ground floor to the roof. Longitudinal steel girders between columns support wood joists at the second floor and roof, and steel beams connect the columns transversely and serve to tie and brace the frame. The lower stories are eventually to have concrete columns surrounding the through steel columns and a flat-slab concrete floor, but the concrete work had not been erected at the time of failure.

Erection of the steel frame was started about Apr. 1 and was pushed rapidly forward. As soon as the frame was riveted the second floor and roof joists and the centering for the first floor of concrete were placed. The wood subfloor for the second floor and the wood sheathing for the roof were placed as soon as the joists were set.

The steel was erected by two stiffleg derricks running on a standard-gage railroad track alongside of the building. Each derrick served one-half the building, or a strip 100 ft. wide. During erection the frame was guyed with steel cables and braced with occasional diagonal wood posts.

On Apr. 10 the erection had progressed about eighteen bays westward, starting at the east end on the south half of the building. On the north half about ten bays had been erected. On the south half the riveting was ten days behind the erection. At about 9:45 a.m. the steel-erection crew was driven to shelter by a heavy storm. At that time the work erected consisted of parts of two bays, which

were guyed with hemp lines. During the shower a heavy squall blew down the newly erected bents toward the north, carrying with them, successively, one bent after another until the riveted section was reached. This remained standing, as did also all of that part of the north half of the frame that had been erected.

The steelwork left standing was racked somewhat, but was immediately drawn back to place and plumbed up. The fallen steelwork was disconnected and cleaned away. Many members, especially the columns, required strengthening. Only a small percentage were so injured as to require replacement. Erection was resumed after three days' delay.

The building was designed and the erection supervised by Mills, Rhines, Bellman, and Nordhoff, architects. The steel erection was carried on under a general contract by the A. Bentley & Sons Co., of Toledo. [Information from the architects.]

Apulia Aqueduct in Use

Water was turned into the Apulia Aqueduct on Apr. 13. The aqueduct and its many branch lines have a combined length of about 1800 miles and will supply water from the River Sele to 3,000,000 people living in numerous cities and towns located in several arid provinces in southeastern Italy. (See ENGINEERING NEWS, July 16, 1914, for map and description.) The project is not yet completed.

Municipal Belt-Line Railway, Oakland, Calif.

Work was begun Apr. 8 on laying a spur track along the new quay wall of the inner harbor at Oakland, Calif. This is the first unit of a municipal belt-line steam railroad that will connect every dock and railroad terminal with the wholesale warehouse district.

For the present the tracks being laid will connect with the Southern Pacific Co.'s lines. Engines and cars of that corporation will be used by the city until the belt-line railroad is completed, when a full equipment will be acquired by the city, and the municipal road will be connected with the Santa Fé, the Western Pacific, the Key Route, the Oakland, Antioch & Eastern, and other lines that enter the city.

The work is to be completed as soon as possible, that all may be in readiness for the opening to business of the first unit of the estuary quay wall on June 1. The foundation is about to be laid for the first of the city's big warehouses, which are to line the municipal water front.

An Investigation of Engineering Education

The Carnegie Foundation for the Advancement of Teaching, in coöperation with the Joint Committee on Engineering Education, is making a study of the methods pursued in American engineering schools. The societies represented on the joint committee are the national societies of Civil, Mechanical, Mining and Electrical Engineers, the Society for the Promotion of Engineering Education, the American Chemical Society and the General Education Board. The investigation is being carried on under the general direction of Dr. Henry F. Pritchett, president of the Carnegie Foundation, with Prof. C. R. Mann, professor of physics in the University of Chicago, in charge.

A set of questions has just been sent out to a large number of engineers. Some of the important questions asked are as follows: What are the particular points at which the work of the engineering schools is effective and successful, and in what is it least effective and successful? What are the most important factors determining probable success or failure in engineering? What sort of work and what kind of experience are most effective in testing a man's fitness to succeed in engineering? Is a reading knowledge of French, German or Spanish essential to an engineer? Make specific suggestions for the improvement of engineering education.

Rebuilding Belgian Cities

Several American engineers have been consulted by a committee of Belgian architects as to the most satisfactory method of reconstructing their ruined cities. M. M. O'Shaughnessy, City Engineer of San Francisco, recently received a letter from Chas. Petris, a Belgian architect, stating that plans were already being perfected for rebuilding, and asking advice on the best type of temporary structures. The method adopted by San Francisco in rebuilding after the disaster of 1906 is of interest to the Belgians and will probably be adopted for some of their cities.

Bills Passed to Protect New Jersey Shore

New Jersey shore property will receive a greater measure of protection by three bills which were recently passed by the State Legislature and approved by the Governor. The first bill, Senate No. 135, authorizes the governing body of any county in New Jersey bordering on the Atlantic Ocean to raise and appropriate money, to be expended by and under the direction of the Secretary of War, for the erection and construction of sea walls, bulkheads, groins, etc., for the protection against encroachments of the sea on inland waterways, navigable rivers and waterways, riparian lands, public roads and boulevards, and public properties. Senate Bill No. 136 is an act permitting the appropriation from the state funds of a sum of money to be expended in the same way and for the purpose just described in the foregoing. The third bill, Senate No. 137, authorizes the governing body of any borough, city, township or other municipality in any county bordering on the ocean to raise and appropriate money to be expended in the same manner and for the same purpose as described by the two other bills. The three bills were presented by Henry E. Ackerson, Jr., Senator from Monmouth County.

Each of the bills states further that in some sections the coast of New Jersey consists of only a very narrow strip of land, containing homes, hotels, business places, streets and boulevards, situated between the Atlantic Ocean and a navigable river, or other inland waters having channels dredged and cared for by the United States Government. From this wording it is evident that the acts are designed primarily for the protection of the Monmouth County shore, and especially that portion in the vicinity of Seabright which for the past two years

has sustained the greatest damage from northeast storms.

Senator Ackerson informs us that he has also drafted a paragraph for the Supplemental Appropriation Bill which provides for an appropriation of $35,000 for making a survey of the shore front and to study the cause of coast erosion from Cape May to Sandy Hook and the preparation of a comprehensive plan for coast protection, the plan to be submitted to the legislature of 1915. No method has as yet been adopted by the state for taking over the proposition of coast protection generally. If the appropriation of $35,000 is allowed, it will be a step in committing the state on this subject.

Deep Diving Experiments and the Records at Honolulu

In raising Submarine F-4, which sank in the harbor of Honolulu, Hawaii, on Mar. 29, 1915, the U. S. Navy Department is for the first time making use of an extensive series of experiments carried on under laboratory and practical conditions during the past two years. In fact, without the benefit of these experiments, it is unquestionable that the submarine would have had to be raised, if at all, by the old-fashioned expedient of dragging.

To Chief Gunner George D. Stillson, U. S. N., belongs a considerable amount of the credit due for the inception of this work. Mr. Stillson was fully conversant with a series of experiments conducted by the British Government in 1906-7, in which he took part himself, as a result of which divers had been able to go down to a depth of 200 ft. with safety. In our own navy, the greatest depth attained was 100 ft. and even that was attended with much risk. The English tests had shown that under the previous methods of diving, divers had been subject to bends upon returning to the surface and that the results had in many cases been fatal. It was shown that this trouble was due to improper decompression; in other words, the diver had been returned to the surface either too rapidly or too steadily. His blood, which had taken up nitrogen from the air, did not have proper opportunity to get rid of it. The English experiments, which were exhaustive, indicated that the best method was to bring the diver up by stages, allowing a period of 15 to 30 minutes at each stop for him to become used to the altered pressure.

After being called to the attention of the Navy Department by Gunner Stillson, the matter was taken up by the Bureau of Construction and Repair in connection with their submarine work, of which Naval Constructor Herbert S. Howard is in charge, and permission was obtained from the Secretary to make extensive tests.

It was early decided that the best results could be obtained first by experiments in a tank where the practical conditions surrounding the diver could be reproduced and he could be kept constantly under observation. The necessity for building such a tank was obviated by the offer of A. Schrader's Sons, Inc., who allowed the Navy Department to use their new tank in Brooklyn, N. Y. In this tank conditions equivalent to depths up to 215 ft. were reproduced by closing the tank and applying pressure up to 100 lb., which was the limit of safety.

The experiments were begun about February, 1914, and continued until the end of the summer. They were under the charge of Gunner Stillson and five chief-gunner's mates. The first part of the time was given up to checking the English decompression tables and to accustoming the party to diving. Later, thorough and exhaustive tests were made of suits, helmets and other diving gear and of various self-contained diving apparatus equipped with chemical and oxygen cylinders, apparatus for submarine escape and any other apparatus of which test was requested.

As a direct result of the experiments, a number of improvements in apparatus and method were developed, among which two are notable. The more important was the development of the first satisfactory method of feeding air to the divers from compressed-air cylinders instead of by hand pumps. The latter had proved inadequate because with them the men could not stand the work they had to do when under water and because the air could not be forced down to any considerable depth. This method was developed by Stillson by the use of properly devised valves, air washers and the like to regulate the supply of air. The other important improvement, also developed by Stillson, was the first feasible diver's telephone, adapted from combinations of different telephones. The effect of such an instrument is far-reaching in increasing the confidence of the diver. Conversation is now possible at any time, where formerly he had to depend on signals by a line.

After the conclusion of these tests, they were supplemented by actual tests in deep water. In November, 1914, the torpedo boat destroyer "Walke" was equipped with a complete outfit of diving apparatus, including a recompression tank, and took the same party to Long Island Sound in the neighborhood of New London. This recompression tank was the first used in the United States for diving purposes and consists of a tank similar in principle to the large one in which the original experiments took place. If a diver upon reaching the surface is affected by bends, he can be immediately placed in the tank and recompressed, then returned by slow stages. It was during these practical tests that Chief Gunner's Mate Drellishak established the world's record of 274 ft. and remained under long enough to do work. It is remarkable that throughout both tests, when diving was being carried on at depths greater than ever before, no accident occurred and there were only two or three mild cases of bends.

Immediately upon the receipt of the news that Submarine F-4 had sunk Chief Gunner Stillson and his five assistants, accompanied by Passed Assistant Surgeon Geo. R. W. French, U. S. N., were placed upon a train and rushed to the Coast, where the U. S. S. "Maryland" was awaiting them. The trip to Honolulu was made in less than 10 days from New York, breaking all records. The submarine was found covered by about 275 ft. of water and the old inadequate method of sweeping was being employed, though one diver had gone down 215 ft., of course without result. The submarine has hoisting pads, and also a hoisting sling, which it is the object of the divers to find in order that cables may be attached. It may also be possible to find the air connection on the submarine and by use of compressed air blow out some of the water in the boat. The divers are unable to stay down for longer than 15 to 20 min. on account of the danger of oxygen poisoning. Up to April 17, three lines had been attached and the greatest depth attained by any diver had been 288-ft., a world's record.

The Commissioner-Manager Plan of city government has been adopted by Grand Rapids, Mich.

A Timbering Collapse at the Twin Peaks Tunnel, San Francisco, Calif., on Apr. 10, killed one workman and injured eight others.

Under-Sea Coal Galleries Collapsed in Japan at the Ube coal mine in the Shimonoseki district, which is at the southwest extremity of the Island of Hondo.

A Suspension-Bridge Cable Broke at Olean, N. Y., on Apr. 18, causing the death by drowning of three persons who were leaning upon it. The bridge spans Olean Creek.

Concrete Forms Were Blown Down on the Brooklyn-Brighton viaduct at Cleveland, Ohio, by a heavy gale on Apr. 11 and several workmen carried down to the ground, 25 ft. below. Four of the men were seriously injured.

A Railway Collision in Detroit, Mich., on April 14, between a street car and a string of freight cars pushed by a switch engine on the Detroit, Toledo & Ironton R.R., killed 15 persons and injured about 20. As the car approached the railroad crossing it stopped and the conductor ran ahead to see if the tracks were clear. It is alleged that he signaled the motorman to wait, but the latter apparently misunderstood the signal, for he ran the car rapidly onto the railroad tracks.

Imhoff Tanks for Sewage Treatment at Beaver Falls, Canonsburg and York, Penn., are included in plans recently approved by the State Department of Health. At Beaver Falls, disinfection is proposed.

The Pecos River in New Mexico was in record flood during the early part of this week, a flow of approximately 70,000 cu.ft. per sec. being observed near Roswell. The east embankment of Lake MacMillan was submerged, and leaks were reported in MacMillan Dam. The Avalon Dam and the Hondo Reservoir are reported safe, but there has been some damage to the canal in the last-named project.

Damages for Typhoid Fever attributed to the water supplied by the Mount Holly (N. J.) Water Co. were awarded by a lower court some time ago and were sustained in an opinion of the State Supreme Court handed down on Apr. 15. The verdict ($750) was in favor of B. Henry Jones, for expenses incurred on account of his three children, who suffered from typhoid during an outbreak in 1912.

Specifications for Culverts and Short-Span Concrete Bridges have been issued by the State Highway Department, Bureau of Township Highways, of the State of Pennsylvania. These specifications are intended primarily for use in connection with plans furnished by the Bureau of Township Highways to township supervisors, but they might prove useful to engineers engaged in similar work. Copies may be obtained by writing to the highway department at Harrisburg, Penn.

Express Companies in the United States in the year ending June 30, 1913, according to the statistics collected by the Interstate Commerce Commission, had gross receipts of $168,880,000, of which $83,873,000 was paid to the railways for express privileges and $79,215,000 was paid out in operating expenses. This left a net operating revenue of $5,792,000, of which $1,379,000 was paid out in taxes, leaving net earnings of $4,353,000 (including $60,000 of other profits). The total value of real estate and equipment owned by the express companies was $33,400,000.

Land Reclamation and Paper Pulp Manufacture—In conjunction with the reclamation of hundreds of thousands of acres of tule land in the lower Sacramento Valley, California, the proposal has just been made by a chemist of the U. S. Bureau of Plant Industry to utilize the tules now growing on the land for the manufacture of paper pulp. It is claimed that 20,000 tons of the marsh reeds can be used annually, that the pulp manufactured from them is of excellent quality and that the revenue therefrom will materially diminish the cost of reclaiming the land.

A Pan-American Road Congress, under the joint auspices of the American Highway Association and the American Road Builders' Association, is to be held at San Francisco during the week beginning Sept. 13. This date, which was decided on Apr. 16 at a meeting of the joint committee representing the two associations, will enable engineers from the East to attend this congress and also the International Engineering Congress which convenes the week following. Plans for the congress are in charge of an executive committee, of which Hon. Chas. W. Gates, Governor of Vermont, is chairman.

The Elmira, N. Y., Water-Works are to be purchased by the city from the Elmira Water, Light & R.R. Co., the recent vote of the municipality being favorable. The agreed price is $1,500,000, based on an appraisal made a year ago by Henry Floy, Consulting Engineer, New York City. The system includes a pumping station, filter plant, four reservoirs and the distributing mains. The funds have been raised by the sale of 4½% municipal bonds, which have been taken by Clark, Dodge & Co., of New York, and are being resold at prices which yield from 4.05 to 4.22%, depending on the length of the term.

Viaduct Started at Tacoma This Month—The construction of a new viaduct was begun on Apr. 8 at Tacoma, Wash. It will be built by the Great Northern Ry. and the Oregon-Washington Ry. & Navigation Co. across switching tracks near Puyallup Ave. The contractor is the Weymouth Construction Co. The viaduct and elevated portion of the approaches will be paved with asphalt. The grade on the approaches will be 6%; the roadway will be 24 ft. wide, with an additional 6-ft. width on one side for a sidewalk. The cost, estimated at $30,000, will be borne by the companies which operate on tracks under the viaduct. At the present time there are 32 tracks at the site, and more will be added. It is planned to make this the freight district of the city.

Coming Civil-Service Examinations—The New York State Civil-Service Commission will hold examinations on May 15, 1915, in various cities throughout the state for the following positions: Landscape Gardener and Superintendent of Parks, State Reservation Commission, Saratoga Springs; $100 per month and traveling expenses. Inspector of Construction, State Department of Highways, counties of Allegany, Broome, Cattaraugus, Chemung, Columbia, Chenango, Clinton, Cortland, Delaware, Essex, Franklin, Fulton, Genesee, Greene, Hamilton, Herkimer, Jefferson, Lewis, Madison, Montgomery, Oneida, Ontario, Orleans, Otsego, Rockland, St. Lawrence, Schenectady, Schoharie, Schuyler, Suffolk, Steuben, Tompkins, Sullivan, Tioga, Washington and Wyoming; salary, $3.50 to $4.50 per day. Mine and Tunnel Inspector, Department of Labor; salary, $1200.

160 Miles of the New York State Barge Canal will be in operation when the state canals are opened for the season. The closing of the Barge Canal dam at Crescent is progressing rapidly, and the work has now reached such a point that there is no doubt of its completion in time for the opening. All boats westward and northward bound will enter the new Barge Canal channel at Waterford, and the general office for the issuing of clearances will be established at Waterford on the new terminal dock near Barge Canal Lock No. 2. The old canal, however, from Waterford to Albany will be retained in its present condition. The opening of the season will place in commission more than 160 mi. of the new Barge Canal channel. This will include 100 mi. of new Erie Canal navigation, consisting of 17 mi. between Waterford and Rexford Flats, 20 mi. in Wayne County and the remainder between Rochester and Tonawanda; also 31 mi. on the Champlain Canal between Northumberland and Whitehall and 35 mi. on the Oswego Canal. On the Erie Canal eleven of the new Barge Canal locks will be operated; on the Champlain Canal six, and all of the new locks on the Oswego Canal, being seven in number. While the through Barge Canal channel on the Oswego Canal will be in use, on account of the non-completion of work at various points boats of 6-ft. draft only will be permitted to go through.

PERSONALS

Mr. Nelson Cunliff, Superintendent of Construction of the Park Department of St. Louis, Mo., has been appointed Park Commissioner of that city.

Dr. Charles Edward Amory Winslow, of the New York State Department of Health, has been appointed Professor of Public Health of the Medical School of Yale University.

Mr. P. F. McManus, Superintendent of the Joliet division of the Elgin, Joliet & Eastern Ry., has been appointed General Superintendent of the road. His former office will be abolished.

Mr. M. N. Baker, President of the Montclair, N. J., Board of Health and Editor of "Engineering News," has been appointed by Governor Fielder a member of the recently reorganized New Jersey State Board of Health.

Mr. Clyde Potts, M. Am. Soc. C. E., Consulting Sanitary Engineer, of Morristown, N. J., and New York City, has been appointed by Governor Fielder a member of the recently reorganized New Jersey State Board of Health.

Mr. Richard P. Harvey, formerly Manager of the Culvert Department of the Wheeling Corrugating Co., of New York City, is now Western Representative of the Concrete Form Co., of Syracuse, N. Y., with offices at 140 Ellis St., San Francisco, Calif.

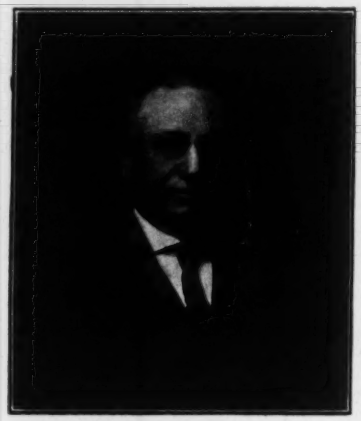
Mr. Clarence Boyle, Jr., Jun. Am. Soc. M. E., District Sales Manager of the Taylor-Wharton Iron & Steel Co., of Scranton, Penn., will become associated, on May 1, with Clarence Boyle, Inc., dealers in wholesale lumber, 1211 Lumber Exchange Bldg., Chicago, Ill.

Mr. Robert F. Hall, who for the past six years has been connected with the Sales Department of the Universal Portland Cement Co., of Chicago, Ill., has succeeded Mr. J. P. Beck as Publicity Manager of the company. Mr. Hall is a graduate of the University of Michigan.

Messrs Edwin S. Jarrett, M. Am. Soc. C. E., formerly Vice-President, and Ralph H. Chambers, M. Am. Soc. C. E., formerly Chief Engineer and General Manager of the Foundation Co., have formed the Jarrett-Chambers Co., Inc., Contractors, with offices at 30 East 42d St., New York City.

Mr. Jesse G. June, Superintendent of Terminals of the Erie R.R., at Jersey City, N. J., has been appointed Superintendent of the Allegheny and Bradford divisions, with offices at Salamanca, N. Y. Mr. Eugene R. Allen, Assistant Superintendent of Terminals, at Jersey City, succeeds Mr. June.

Mr. Samuel Tobias Wagner, M. Am. Soc. C. E., whose promotion to be Chief Engineer of the Philadelphia & Reading Ry. was announced in our personal columns of Apr. 15, was born in Philadelphia in 1861 and graduated from the University of Pennsylvania in 1881, when he was awarded the senior prize in civil engineering. In 1884 he received a degree of Civil Engineer from his alma mater. His early experience in engineering was as draftsman, Assistant Superintendent of Shops and Superintendent of Shops of the Phœnix Iron Co. from 1881 to 1893, except for about a year when he was Resident Engineer of Erection on the construction of the Louisville & Jeffersonville Bridge over the Ohio River for the Phœnix Bridge Co. in 1890. In 1893 and 1894 Mr. Wagner was Chief Draftsman of the bridge division of the Bureau of Surveys of the City of Philadelphia. From 1894 to 1900 he was Assistant Engineer in charge of the design and construction of the Pennsylvania Ave. subway and tunnel of the Philadelphia & Reading Ry. for the Philadelphia Bureau of Surveys, work involving the lowering of tracks of the railway and costing about $6,000,000. From 1900 to 1902 he was Assistant Engineer in charge of the improvement, extension and filtration of the water-supply of Philadelphia, during which time most of the present filtration plant was designed and placed under contract. From 1902 until the present time Mr. Wagner has been with the Philadelphia & Reading Ry. as Assistant Engineer on grade-crossing work, engaged chiefly on grade-crossing elimination work in the vicinity of Philadelphia, which involved a total cost of about $10,000,000. He is a well-known contributor to the "Transactions" of the American Society of Civil Engineers, having written six papers, for one of which, in 1915, he was awarded the James Laurie prize. He is also a member of the American Society for Testing Materials and is a Past-President of the Engineering Alumni Society of the University of Pennsylvania. He is a trustee, Professor of Engineering and president of the faculty of the Wagner Free Institute of Science of Philadelphia.



Samuel T. Wagner

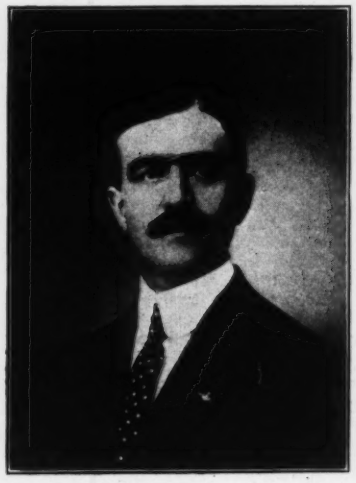
Messrs. John S. Griggs, formerly of the consulting firm of Griggs & Holbrook, of New York City, and David Moffat Myers, formerly Mechanical Engineer of the United States Leather Co. and more recently in private consulting practice, announce the consolidation of their practice under the firm name of Griggs & Myers, Consulting Engineers, 110 West 40th St., New York City.

Mr. Robert J. Cunningham, Comptroller of Allegheny County has been appointed State Highway Commissioner of Pennsylvania by Governor Brumbaugh, to succeed Mr. Edward M. Bigelow, M. Am. Soc. C. E., recently resigned. Mr. Cunningham was born at Elizabeth, Penn., in 1860, and received his education in the public schools of Pittsburgh and the academy at Sewickley. He was a member of the Sewickley Council for seven years and served as Comptroller of Allegheny County from 1903 to 1906. In the latter year he was elected to the State Senate. In 1908 he resigned and again became Comptroller of Allegheny County, which position he has filled up to the present time.

Maj. Gen. Wm. C. Gorgas, U. S. A., has been invited by the Rockefeller Foundation to become a permanent member of its staff in the capacity of general adviser in matters relating to public sanitation and the control of epidemics. It is stated by the foundation that there is no basis for the published reports that General Gorgas is to receive a salary of $50,000, but that the offer includes a moderate salary and the assurance of the usual allowance in the event of resignation or death. By act of Congress General Gorgas is entitled to retire from the army at any time on three-quarters pay. In the event of his accepting the foundation's offer he will probably resign from the army and go at once to Servia to direct the campaign there against typhus.

Mr. John Meigs, M. Am. Soc. C. E., Assistant Director of the Philadelphia Department of Wharves, Docks and Ferries, has been made Director of the Department. Mr. Meigs is a graduate of Columbian University, class of 1894. After graduation he entered the employ of the Capital Traction Co., of Washington, D. C., as a draftsman, and later held a similar position with the Union Bridge Co., of Athens, Penn. In 1898 he entered the government service as Junior Engineer in the U. S. Engineer Office, remaining eight years, after which he was engaged by the Penn Bridge Co. as Engineer and Superintendent in charge of construction of the Washington Channel Bridge, the Anacostia Bridge and other bridges in that city. In 1907 Mr. Meigs went into private contracting business, but the following year returned to the government service, subsequently being appointed Assistant Engineer in charge of the dredging operations in the Delaware River.

W. D. Uhler, M. Am. Soc. C. E., whose appointment as Chief Engineer of the Pennsylvania State Highway Department was announced in our issue of Apr. 8, was born at Nazareth, Penn., Nov. 8, 1872. From 1890 to 1895 he was connected with the Engineering Department of the Lehigh Valley R.R. For eight years thereafter he was with the Queen Anne's R.R. in various capacities from Assistant Engineer to General Manager and Chief Engineer. From 1904 to 1908 Mr. Uhler was County Roads Engineer of Caroline County, Maryland, from which position he was promoted to be Engineer in Charge of Maintenance of the Maryland State Roads Commission. This position he resigned in 1912 to become Principal Assistant Engineer of the Bureau of Highways of the City of Philadelphia. He is a member of the American Road Builders' Association, the Engineers' Club of Philadelphia and the Engineers' Club of Baltimore.



W. D. Uhler

OBITUARY

Eli A. Miller, Superintendent of Motive Power of the New York, Chicago & St. Louis R.R., died Apr. 17 at his home in Cleveland from pneumonia. Mr. Miller was born in Washington County, Pennsylvania. He entered the railway service as an employee of the Canadian Pacific Ry. In 1882 he became Master Mechanic of the New York, Chicago & St. Louis, and in 1905 was appointed Superintendent of Motive Power, which position he held up to the time of his death.

Daniel R. Russell, President of the D. R. Russell Engineering & Development Co., of St. Louis, Mo., which he founded about a year ago, died suddenly in New York City, Apr. 9, from heart disease. Mr. Russell was born and educated in St. Louis. After graduation from Smith Academy, in that city, he entered his father's business, the Parker-Russell Mining & Manufacturing Co., and ten years later became president of the company. Mr. Russell was 50 years of age. His widow and a son survive him.

Fred C. Keighley, M. Am. Inst. M. E., General Superintendent of the Oliver & Snyder Steel Co., Oliver, Penn., was found dead in his garage on the night of Apr. 14. It is believed that he shot himself. He was for six years President of the American Institute of Mining Engineers. He was formerly an inspector for the State Mining Department of Pennsylvania, and had considerable to do with the development of the Connellsville coke region. Mr. Keighley was 67 years of age. He left a widow and five children.

John M. Sherrerd, M. Am. Soc. C. E., died at his home in Easton, Penn., Apr. 15, 1915. Mr. Sherrerd was a graduate of Rensselaer Polytechnic Institute. At the time of his death he held the position of Assistant to the President of the Kennedy-Stroh Corporation, Pittsburgh, Penn. Before its absorption by the aforementioned concern, he was Secretary of the Stroh Steel Hardening Process Co., of Pittsburgh. Prior to his connection with the Stroh interests, Mr. Sherrerd was for 18 years the Sales Manager of the Taylor-Wharton Iron & Steel Co., High Bridge, N. J.

Henry Robinson, M. Inst. C. E., for 22 years Professor of Civil Engineering at King's College, died at 78 years of age at his home in Abbey Road, N. W., London, England, Mar. 24. After graduation from King's College, Professor Robinson joined the staff of Sir William Armstrong, at Elswick, after which he practiced as a civil and consulting engineer at Westminster. Among the many important projects which he developed during 50 years of practice was the design and construction of the hydraulic-power plant at Hull, the first scheme of the kind in the kingdom.

Henry W. Poor, President of Poor's Railroad Manual Co., died Apr. 13, at his home in New York City in his 72d year. He was born in Bangor, Me., and was the son of Henry V. Poor, who was editor of the "American Railroad Journal," now the Mechanical Edition of "Railway Age Gazette." After graduation from Harvard University in 1865, Mr. Poor became a partner with his father in the banking and publishing firm of H. V. & H. W. Poor, and the same year the first number of the "Manual" was published. Until the establishment of the statistical office of the Interstate Commerce Commission, in 1887, the "Manual" was the only source of detailed information concerning the history and financial affairs of the railroads of the United States.

William Harlin Kennedy, M. Am. Soc. C. E. since 1871, died at Pittsburgh, Penn., Apr. 14, 1915, aged 80 years. During his early life he was connected with the Pennsylvania R.R., also serving in the Pittsburgh & Connellsville R.R. under Mr. Sigismund Low, with whom he was associated as United States Assistant Engineer on the survey of the Ohio River, under Col. W. Milnor Roberts, former President of the American Society of Civil Engineers. Later on Mr. Kennedy was Chief Engineer of the New Pittsburgh Water Works, building the Highland, Brilliant Hill and Herron Hill reservoirs, as well as laying much of the present water-pipe distribution system. He was for a long time Chief Engineer of the Oregon-Washington R.R. & Navigation Co., stationed at Portland, Ore. Later, he was associated with Virgil G. Bogue, M. Am. Soc. C. E., on the Western Pacific R.R. During the latter part of his life he lived in Pittsburgh, near which city he was born.

Charles William MacCord, Professor Emeritus of Stevens Institute of Technology, died at his home in Hoboken, N. J., Apr. 13, at 79 years of age. Professor MacCord was born in Dutchess County, N. Y., and was educated at Princeton University. After graduation he acquired a sufficient knowledge of mechanical drawing to enable him to secure a position as assistant draftsman at the old De Lamater Iron Works, of New York City, where his work attracted the attention of Capt. John Ericsson, who engaged him as his chief draftsman. After nine years spent in Capt. Ericsson's employ, during which time he drew the plans for the "Monitor" of Civil War fame, Professor MacCord went to Hoboken in 1868, as chief draftsman upon the Stevens battery, then being constructed under General McClellan, and later held a similar position in the Department of Docks of New York City. With the opening of Stevens Institute in 1871 he accepted the chair of Mechanical Drawing, which he filled until 1906, when he retired and became Professor Emeritus. He was the author of numerous writings on higher mathematics.

ENGINEERING SOCIETIES

COMING MEETINGS

AIR BRAKE ASSOCIATION.
May 4-7. Convention at Chicago. Secy., F. M. Nellis, 53 State St., Boston, Mass.

AMERICAN WATER WORKS ASSOCIATION.
May 10-14. Annual meeting in Cincinnati. Secy., J. M. Diven, 47 State St., Troy, N. Y.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 11-13. Annual meeting at Hotel Astor, New York City. Secy., Franklin H. Wentworth, 87 Milk St., Boston, Mass.

CONFERENCE OF BUILDING INSPECTORS.
May 14. Meeting in New York City. Secy., Sidney J. Williams, Madison, Wis.

INTERNATIONAL RAILWAY FUEL ASSOCIATION.
May 17-20. Annual meeting at Chicago. Secy., C. G. Hall, 922 McCormick Bldg., Chicago.

SOUTHWESTERN ELECTRICAL AND GAS ASSOCIATION.
May 19. Annual convention at Galveston, Tex. Secy., H. S. Cooper, Dallas, Tex.

NATIONAL MACHINE TOOL BUILDERS' ASSOCIATION.
May 20-21. Spring meeting at Atlantic City, at Hotel Marlborough-Blenheim. Secy., Charles E. Hildreth, Worcester, Mass.

NATIONAL CONFERENCE ON CITY PLANNING.
June 7-9. National Conference in Detroit, Mich. Secy., Flavel Shurtleff, 19 Congress St., Boston, Mass.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 9-11. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

MASTER CAR BUILDERS' ASSOCIATION.
June 14-16. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
June 17-18. Semi-annual meeting at Toledo. Secy., Frank E. Sanborn, Ohio State University, Columbus.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 22-25. Annual meeting at Iowa State College, Ames, Iowa. Secy., F. L. Bishop, University of Pittsburgh, Pittsburgh, Penn.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 22-25. Summer meeting at Buffalo, N. Y. Secy., Calvin W. Rice, 29 W. 39th St., New York City.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 22-26. Annual meeting in Atlantic City. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

Illinois Association of County Superintendents of Highways—This association has been organized recently and includes engineers who hold positions as county superintendents of highways. President, R. W. Osborn, Pontiac, Ill.; Secretary, Walter E. Emery, Peoria, Ill.

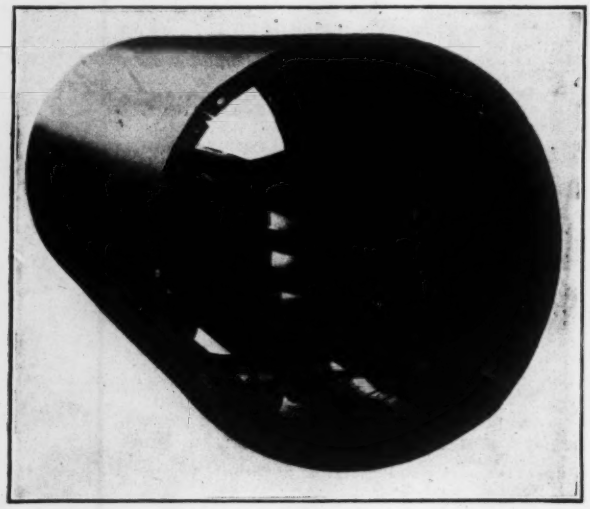
Detroit Engineering Society—At the annual banquet of the Detroit Engineering Society, to be held on Apr. 24, President Maclaurin, of the Massachusetts Institute of Technology, will be the principal speaker, his subject being "Broadening the Profession of Engineering." Among the other speakers of the evening will be W. F. M. Goss, who will speak on "A Modern Engineering Problem"; Francis C. Shenehon, "Men of Energy—Soldiers, Sea Captains and Civil Engineers"; George W. Patterson, "Ideals for Engineers"; George W. Bissell, "Some Problems in Teaching Engineering"; Warren S. Blauvelt, "An Engineer's Duty to His City"; Lieut.-Col. Mason W. Patrick, "One Phase of Government Work." H. H. Esselstyn, the retiring President, will act as toastmaster. The Detroit Engineering Society is 21 years old, and its present membership is about 650, having increased over 200 in the past year. The Secretary is Frederick H. Mason, 614 Moffatt Bldg.

American Society of Mechanical Engineers—Two funds of $1000 each, known as the Hess Prize Funds, have been established by the Society through the generosity of its vice-president, Henry Hess. The income from the funds may be awarded annually for the best paper by a junior member and to the two enrolled student-branch members who contribute the best papers. These must be submitted by June 30, 1915, for award to be made at the annual meeting in December. The junior prize consists of $50 in cash and an engraved certificate signed by the president and secretary of the Society. The student prizes will consist of $25 each, with engraved certificates. The prizes will be awarded only if the papers are adjudged to be of sufficient merit as contributions to the literature of the profession of mechanical engineering. If in any one year no award is made, the accumulated prizes may be awarded to authors in other years if their papers are deemed of sufficient merit.

Appliances and Materials

Demountable Steel Sewer Forms

The steel sewer forms shown in the accompanying view have just been put on the market by the Blaw Steel Construction Co., Pittsburgh, Penn. They are being extensively used on some sewer contracts in San Antonio, Tex. The form is made up of four sheet-steel quarter-cylinders (standard in size for any given sewer diameter), with the sections identical in design and dimensions except that the bottom section is provided with outstanding angles which form a track on which a truck may run. The full section is erected by bolting the flanges and inserting the wedge-shaped wood stringer shown



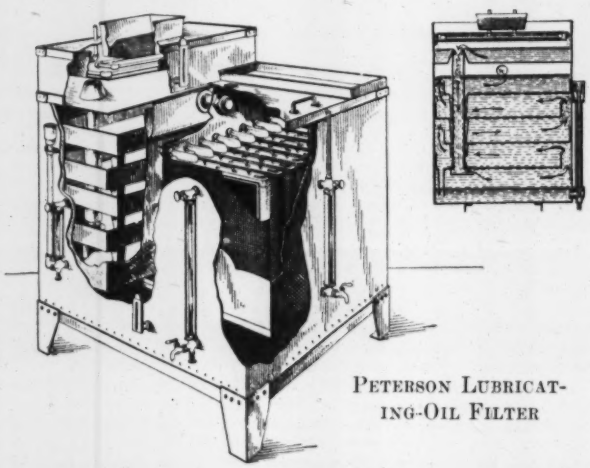
A New Take-Down Sewer Form

at the right of the view. In taking down, the bolts and wood strips are removed, and the sections are placed on the truck and moved down the already erected forms to the end of the framework, where erection proceeds as before.

* * *

Lubricating-Oil Filter

An oil filter of great capacity for the space occupied has recently been put out in commercial form by the Richardson-Phœnix Co., of Milwaukee. The machine treats in five steps the used oil returned to it: (1) Heating to lower the viscosity, (2) flow over a series of shallow trays to precipitate water and dirt, (3) filtration, (4) storage and (5) cooling. The oil passes, as may be seen in the accompanying sketches, through a strainer box and heating tray, then down a funnel and pipe to the lower precipitation trays, up over these and out to the filter chamber. The water collected by each tray is bypassed by the funnels around the downpipe and it collects in the bottom of the oil chamber, from which it is forced out to the waste line through an overflow pipe with adjustable top. The filter frames are immersed in the oil, which flows inward from the outside and thence through nozzles into the clean-oil storage chamber. The nozzles fit spring valves so that filter units can be removed for cleaning without interfering with continuous operation. The frames are designed to prevent folds in the filter cloth; the collection surfaces are vertical to promote self-cleaning.

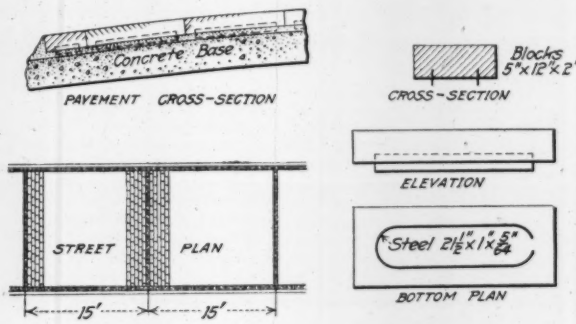


Peterson Lubricating-Oil Filter

* * *

Noncreeping Asphalt Blocks

A possible means of preventing the creeping of asphalt blocks in the direction of traffic, such as was described in "Engineering News," July 16, 1914, is now used extensively by the Hastings Pavement Co., of New York City. After many experiments with blocks having various kinds of projections on the under surface and with iron plates or ribs set in the mortar bed, the type of "anchor" block shown in the accompanying illustration has been adopted. The use



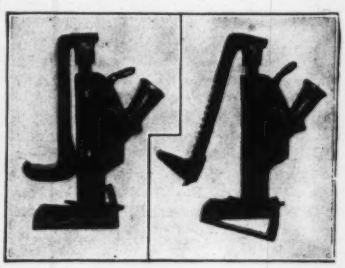
Anchor Block for Asphalt Pavements

of the blocks is fully explained in the lower sketch. On very wide streets, such as Ocean Ave., Brooklyn (referred to in "Engineering News," July 16, 1914), in addition to courses across the whole width of the pavement, two intermediate courses extending from the curbs only one-fourth or one-third across the street would usually be placed between each two whole courses.

* * *

Adjustable Jack with Tilting Base

The special features of the jack shown herewith are the tilting shoe, or base plate, and the adjustable lifting bar.



Jack with Tilting Base and Adjustable Lifting Bar

The tool can be tilted either forward or sideways, and can be held at any desired angle while having a substantial footing. The lifting bar can be placed at the front or side of the jack; a special bar has an adjustable shoe, or hook, which can be set at any height on the bar so as to engage a load that is too low to be lifted by the head of the jack, and at the same time to allow the tool to give its full lift. The device is a specialty of the Buckeye Jack Mfg. Co., Alliance, Ohio. It is 22 in. high and weighs 100 lb.; the carrying capacity is 15 tons and the lift is 10½ in.

* * *

Rust Protection for Steelwork

The Dover Boiler Works, 50 Church St., New York City, announces that it has undertaken the general application of the so-called "Bitumastic enamels" to its steel-plate products, guaranteeing protection for ten years. It will be recalled that these paints are asphaltic compounds which have been found of great use in marine service; their use for land structures has been more limited.